PROCEEDINGS

OF THE

AMERICAN SOCIETY

OF

CIVIL ENGINEERS

VOL. XLIII—No. 6

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(INSTITUTED 1852)

VOL. XLIII—No. 6

AUGUST, 1917

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NEW YORK 1917

American Society of Civil Engineers

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ON MATERIALS FOR ROAD CONSTRUCTION: W. W. Crosby, A. W. Dean, H. K. Bishop, A. H. Blanchard, George W. Tillson, Nelson P. Lewis, Charles J. Tilden.

TO CODIFY PRESENT PRACTICE ON THE BEARING VALUE OF SOILS FOR FOUNDATIONS, ETC.: Robert A. Cummings, Edwin Duryea, E. G. Haines, Allen Hazen, James C. Meem, Walter J. Douglas.

ON THE REGULATION OF WATER RIGHTS: F. H. Newell, W. C. Hoad, John H. Lewis.

TO REPORT ON STRESSES IN RAILROAD TRACK: A. N. Talbot, A. S. Baldwin, G. H. Bremner, John Brunner, W. J. Burton, Charles S. Churchill, W. C. Cushing, Robert W. Hunt, George W. Kittredge, Paul M. LaBach, C. G. E. Larsson, G. J. Ray, Albert F. Reichmann, H. R. Safford, F. E. Turneaure, J. E. Willoughby.

The House of the Society is open from 9 A. M. to 10 P. M. every day, except Sundays, Fourth of July, Thanksgiving Day, and Christmas Day.

HOUSE OF THE SOCIETY—220 WEST FIFTY-SEVENTH STREET, NEW YORK.

TELEPHONE NUMBER........................1446 Circle.
CABLE ADDRESS........................"Ceas, New York."

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AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PROCEEDINGS

This Society is not responsible for any statement made or opinion expressed in its publications.

SOCIETY AFFAIRS

CONTENTS

MINUTES OF MEETINGS

OF THE SOCIETY

May 16th, 1917.—The meeting was called to order at 8.30 P. M.; President George H. Pegram in the chair; Chas. Warren Hunt, Secretary; and present, also, 89 members and 8 guests.

A paper by J. F. Partridge, Jun. Am. Soc. C. E., entitled "Modern Practice in Wood Stave Pipe Design, and Suggestions for Standard Specifications", was presented by the Secretary, who also read communications on the subject from Messrs. H. von Schrenk, F. F. Bell, H. D. Coale, and D. C. Henny. The paper was discussed further by Messrs. Henry P. Rust, H. B. Machen, W. J. Boucher, and F. M. Robbins.

The Secretary announced the election of the following candidates on May 15th, 1917:

As Members

Charles Rogy Burky, Jerome, Idaho
Albert Bennett Cudebec, New York City
Elmer Ellsworth Greenwood, Skowhegan, Me.
Henry Guttin, New York City
Harry Allardt Kluegel, Denver, Colo.
Lamar Lyndon, New York City
Hisanori Numata, Yawata-Machi, Chikuzen, Japan
Jacob Latch Warner, Wilmington, Del.
Ralph Nims Whitcomb, New York City

As Associate Members

Murray Chase Ayers, Pasadena, Cal.
William Earnest Baldry, Dodge City, Kans.
John Ralph Bogert, Wilkinsburg, Pa.
George Hathaway Canfield, Oakland, Cal.
Earl William Chamberlin, Chicago, Ill.
Joseph Brandly Converse, Montgomery, Ala.
Daniel Boyden Cutler, Tyler, Tex.
Lott Davis Draper, Germantown, Pa.
Arthur Hoyt Dunlap, Barstow, Tex.
Lester Fisher Ellis, Lexington, Mass.
Lambert Theodore Ericson, Toledo, Ohio
Chester McKenzie Everett, New York City
Rudolph Evers, Brooklyn, N. Y.
Alvin Bartholdi Fox, Perth Amboy, N. J.
Charles Grover Gabelman, Fort Mills, Philippine Islands
Leslie Drew Goddard, Ann Arbor, Mich.
William Robert Goodwin, Minneapolis, Minn.
Howard Allison Gray, West Somerville, Mass.
Clarence Saylor Gruetzmacher, Milwaukee, Wis.
Benjamin Mortimer Hall, Jr., Atlanta, Ga.
Aloysius Frank Harter, Phœnix, Ariz.
Eugene Robert Hoffman, Olympia, Wash.
Joseph Earl Huber, Mt. Vernon, Ill.
William Hill Hulsizer, Omaha, Nebr.
Hollister Johnson, Albany, N. Y.
Percival Charles Jones, Cleveland, Ohio
Robert Leroy Jones, Sacramento, Cal.
Gerald Sturtevant Kibbey, Minneapolis, Minn.
Paul Robert Kirstein, Cincinnati, Ohio
Enno Paul Knollman, Columbus, Ohio

Adonis William Kreamer, Wheeling, W. Va.
Edward Allyn Lambert, Bridgeport, Conn.
Arthur Lewis LaRoche, Binghamton, N. Y.
Morgan Foster Larson, Perth Amboy, N. J.
Percival John MacNaughton, Springfield, Mass.
William Oliver McCluskey, Jr., Wheeling, W. Va.
Hyman Harry Mandelzweig, Cleveland, Ohio
William Christian Miller, Sycamore, Ill.
James Shively Neibert, Liberty, Tex.
George Frances Nicholson, Seattle, Wash.
Albert Roy Olds, Havana, Cuba
Frank Hugh O'Rourke, Sellersville, Pa.
Edmund Addison Pratt, Philadelphia, Pa.
Maurice James Quinn, Detroit, Mich.
Charles Ramsay Rinehart, New York City
Wilbert Robin Rosche, Pueblo, Colo.
Charles Christian Scharpenberg, Bakersfield, Cal.
James Hilton Sherman, Kansas City, Mo.
Elmer Sigler, Kansas City, Mo.
Isaac Cleveland Steele, Alameda, Cal.
Otto Christian Tretten, San Luis Obispo, Cal.
William Claude Tyson, Little Rock, Ark.
Thomas Albert Van Amburgh, Oklahoma, Okla.
Claude Linn Van Auken, Chicago, Ill.
George Parker Van Vliet, Cincinnati, Ohio
Lester Carl Walker, Richfield, Idaho
Herbert Kirkman Ward, San Diego, Cal.
Lee Gilbert Warren, Milwaukee, Wis.
Harry Artemas Wells, Hanover, N. H.
Herbert James Wright, Philadelphia, Pa.

As Juniors

Walter David Binger, New York City
James DeWitt Bohlken, Blacksburg, Va.
Arthur Taylor Bragonier, Roanoke, Va.
John Bernard Breymann, Jr., Toledo, Ohio
Simon Bricklin, Philadelphia, Pa.
Clarence Condon Dustheimer, Cristobal, Canal Zone, Panama
Aaron Herbert Frank, Utica, N. Y.
Roland Russell Graham, Elmira, N. Y.
Bernard Henry Grehan, New Orleans, La.
Harry Neville Jenks, Berkeley, Cal.
George Seymour McCullough, Evanston, Ill.
Thomas Charles Morgan, Brooklyn, N. Y.
David Nabow, Charlotte, N. C.

William Henri Francais August Pockels, Buenos Aires, Argentine Republic
Frederic Borradaile Prichett, Philadelphia, Pa.
George Francis Sexton, New York City
William Edward Shea, Remedios, Cuba
Walter Raymond Weber, Denver, Colo.

The Secretary announced the transfer of the following candidates on May 15th, 1917:

From Associate Member to Member

Farrand Northrop Benedict, East Orange, N. J.
Clarence Moore Blair, New Haven, Conn.
Gustave Maurice Braune, Cincinnati, Ohio
Frederick Charles Carstarphen, Trenton, N. J.
John Percival Davies, New York City
Farley Gannett, Harrisburg, Pa.
Otto Henry Gentner, Jr., Philadelphia, Pa.
Samuel Rowland Ginsburg, La Romana, Dominican Republic
Oliver Zell Howard, Lawrence, Mass.
John Nathaniel Mackall, Harrisburg, Pa.
William Hale Phillips, Spokane, Wash.
Rex Cameron Starr, Los Angeles, Cal.

From Junior to Associate Member

Chester Ely Atwood, Valier, Mont.
Wilbur Earl Bickerton, Baltimore, Md.
Guy Hersey Bishop, Oelwein, Iowa
Frank Leonard Bolton, Erie, Pa.
Edouard Jean Bernard de Mey, Pittsburgh, Pa.
Harold Lewis English, Washington, D. C.
William Wetmore Gibbs, Nichols, Fla.
Ewing Sloan Humphreys, Richmond, Va.
Ernst Gustav Kaufmann, Buffalo, N. Y.
John Bruce Mailey, Lynn, Mass.
Clifford Eaton Murray, Newark, N. J.
Ernest Benjamin Nelson, Chañaral, Chile
Kingsbury Eastman Parker, San Francisco, Cal.
John Joseph Francis Phalan, Utica, N. Y.
Charles Lysander Rakestraw, San Francisco, Cal.
Harold August Hastrup Schultz, Detroit, Mich.
Valeriano Segura, Manila, Philippine Islands
Minton Machado Warren, Cambridge, Mass.
Maurice Anderson Webster, Philadelphia, Pa.

The Secretary announced the following deaths:

STANLEY ALFRED MILLER, of Paducah, Ky., elected Junior, February 4th, 1902; Associate Member, April 6th, 1909; Member, June 24th, 1916; died May 13th, 1917.

DAVID SIMSON, of Hitchin, Herts, England, elected Member, January 8th, 1902; died December 16th, 1916.

JOHN HATFIELD FRAZEE, of New York City, elected Associate Member, December 6th, 1899; died May 4th, 1917.

LEWIS ROBERTS POMEROY, of Orange, N. J., elected Associate, April 2d, 1890; died May 7th, 1917.

Adjourned.

June 6th, 1917.—The meeting was called to order at 8.30 P. M.; President George H. Pegram in the chair; Chas. Warren Hunt, Secretary; and present, also, 196 members and 43 guests.

The minutes of the meetings of April 18th and May 2d, 1917, were approved as printed in *Proceedings* for May, 1917.

E. F. Robinson, Assoc. M. Am. Soc. C. E., Captain, Corps of Engrs., N. G. N. Y., addressed the meeting on "The Work of the 22d N. Y. Engineers on the Mexican Border", illustrating his remarks with lantern slides, and J. H. Granbery, M. Am. Soc. C. E., exhibited French and German helmets, gas masks, and other military accoutrements from the French front at Verdun.

The Secretary read a communication from J. H. Gandolfo, Assoc. M. Am. Soc. C. E., announcing the establishment of an evening section of a course of theoretical and practical instruction in military subjects by the College of the City of New York, during the summer.

The Secretary announced the following deaths:

JAMES WALTER GRIMSHAW, of Westminster, London, England, elected Member, November 7th, 1888; died February 15th, 1917.

FRANKLIN BUCHANAN LOCKE, of North Adams, Mass., elected Member, March 1st, 1893; died May 11th, 1917.

Adjourned.

ELECTIONS AND TRANSFERS BY THE BOARD OF DIRECTION, JUNE 11th-12th, 1917.

ELECTED AS MEMBERS

LYONEL AYRES, Duluth, Minn.
ALFRED WILLIAM BOWIE, New York City
MALCOLM COLBURN CLEVELAND, New York City
WILLIAM CANTRILL CURD, St. Louis, Mo.
DAVID SMITH GENDELL, JR., Pottstown, Pa.
PAUL ANTHONY TROST, West Hoboken, N. J.

Elected as Associate Members

Raymond Ashton, San Francisco, Cal.
Frederick Andrew Baker, Bogalusa, La.
Wilbur Vick Banister, Boston, Mass.
John Robert Biedinger, Cincinnati, Ohio
Robert Lawton Bowen, Providence, R. I.
Robert Platt Boyd, Monroe, La.
Robert Wesley Briggs, Yonkers, N. Y.
John Henry Brown, Jr., Philadelphia, Pa.
Charles Homer Buford, Chicago, Ill.
Isaac Jacques Casey, Jr., Irvington, N. J.
John Hirst Caton, 3d, Providence, R. I.
Charles Walter Chapin, San Luis Obispo, Cal.
Francis Dorsey Christhilf, Baltimore, Md.
George Phelps Clayson, Chicago, Ill.
William Henry Courtenay, Philadelphia, Pa.
James Cresson, Norristown, Pa.
Perry Augustus Fellows, Ann Arbor, Mich.
Clarence Stephens Gale, Easton, Md.
Franklin Joseph Hanmer, North Milwaukee, Wis.
Neal Hanson, Ute Park, N. Mex.
Wellington Prescott Hews, San Francisco, Cal.
Arthur Julius Knight, Worcester, Mass.
Lyman Calvin Lamb, Youngstown, Ohio
Henry Lloyd, Cheyenne, Wyo.
Charles Edward MacLean, Boise, Idaho
Charles Allen Merriam, Portland, Ore.
John Earl Morelock, Chattanooga, Tenn.
Thomas Preston Paxton, Okmulgee, Okla.
William Clay Penn, Alcoa, Tenn.
Victor Smith Persons, San Francisco, Cal.
Percy Lawrence Reed, Philadelphia, Pa.
William Cathcart Riddle, Harrisburg, Pa.
Andrew Francis Ross, Powell, Wyo.
Lewis Pelot Scott, Aurora, Ill.
George Milson Shepard, Minneapolis, Minn.
Eugene Adalbert Silagi, Columbus, Ohio
John William Swaren, Hayward, Cal.
Rock Granite Taber, Dallas, Tex.
Louis Earle Thornton, Pensacola, Fla.
Frank Kline Webb, New Orleans, La.

Elected as Juniors

Roy Leonard Anderson, West Berkeley, Cal.
Alfred Andrew Burger, Akron, Ohio

Charles Michael Carnelli, New York City
Daniel Warwick Colhoun, New York City
Alden Wales Harvey, Coatesville, Pa.
Alfred John Mahnken, Weehawken, N. J.
Arthur Clough Nichols, Knoxville, Tenn.
George Anton Repko, Queens, N. Y.
Leo Francis Reynolds, St. Joseph, Mo.
Fonzie Eugene Robertson, Dallas, Tex.
William Lewis Stanton, Los Angeles, Cal.
William Robert Swanson, Chicago, Ill.
Charles Laurence Warwick, Narberth, Pa.

Transferred from Associate Member to Member

Joseph Chester Allison, Calexico, Cal.
Charles Terrell Bartlett, San Antonio, Tex.
Percy Lewis Braunworth, Montclair, N. J.
Charles Franklin Brown, Salt Lake City, Utah
John Richard Cahill, San Francisco, Cal.
Edwin Keen Cortright, Lawrence, Mass.
George Cromwell, San Diego, Cal.
William Earle Elam, Greenville, Miss.
Henry Dievendorf Dewell, San Francisco, Cal.
William Dewoody Dickinson, Little Rock, Ark.
Franklin Edward Estes, Petersburg, Fla.
Harry Carter Gardner, Lancaster, Pa.
Albert Wesley Gaumer, Santiago de Cuba, Cuba
Howard Howell George, Newark, N. J.
Trygve Daniel Bödtker Groner, Springfield, Ohio
Charles Newton Green, New York City
Samuel Whilden Henderson, Excelsior Springs, Mo.
Clarence Decatur Howe, Port Arthur, Ont., Canada
Walter Leroy Huber, San Francisco, Cal.
Hinman Barrett Hurlbut, Washington, D. C.
Willis Ranney, San Antonio, Tex.
Richard Wood Randolph, Ichang, China
John Marie Thomas Rice, Pittsburgh, Pa.
Percy Augustus Shaw, Lancaster, Pa.
Thomas Shackelford Shepperd, Pittsburgh, Pa.
Walter James Spalding, Ancon, Canal Zone, Panama
Adolphus Gustavus Trost, El Paso, Tex.
Hubert Southwick Tullock, Leavenworth, Kans.
Guy Anderson Watkins, Little Rock, Ark.
Maurice William Williams, Albany, N. Y.
William Horace Williams, New Orleans, La.

TRANSFERRED FROM JUNIOR TO ASSOCIATE MEMBER

RICHARD HENDERSON EURICH, Montclair, N. J.
FRED DAILEY HARTFORD, Denver, Colo.
JOSÉ JUSTO MANZANILLA Y CARBONELL, Havana, Cuba
CHARLES ERNEST RAMSER, Washington, D. C.
NEIL THOM, JR., San Francisco, Cal.
JOHN MALCOLM WALLER, Kansas City, Mo.
GEORGE LELAND YOUMANS, Tacoma, Wash.

OF THE BOARD OF DIRECTION

(Abstract)

May 15th, 1917.—The Board met at 10.30 P. M., immediately after the adjournment of the Membership Committee; President Pegram in the chair; Chas. Warren Hunt, Secretary; and present, also, Messrs. Davies, Fay, Flinn, Humphreys, Kittredge, and Noble.

Ballots for membership were canvassed, resulting in the election of 9 Members, 60 Associate Members, and 18 Juniors, and the transfer of 19 Juniors to the grade of Associate Member.

Twelve Associate Members were transferred to the grade of Member.

A Report from the Membership Committee was received and acted upon.

Adjourned.

SOCIETY ITEMS OF INTEREST

Letter from the United States Coast and Geodetic Survey

"WASHINGTON, D. C., June 5, 1917.

"DR. CHARLES WARREN HUNT,
Secretary, American Society of Civil Engineers,
New York, N. Y.

"DEAR SIR.—I am sending you herewith a copy of the bill recently passed by Congress which includes an item of vital interest to the Coast and Geodetic Survey.

"It authorized the President to turn over to the War or to the Navy Department, in time of war, any part of the equipment or personnel of the Survey that he may think advisable for the national defense and offense.

"The bill also provides that any of the personnel assigned to duty in the War or Navy Department shall have proper military status. The status of the engineers of the Survey is provided for in the bill.

"An item of the greatest importance, not only to this bureau but to civil engineers throughout the country, is the provision that the field engineers, who will be called hereafter hydrographic and geodetic engineers and junior hydrographic and geodetic engineers, and aids, are to be appointed or commissioned by the President of the United States, with the advice and consent of the Senate.

"Those persons in the Survey who have had the title of Assistant and of Aid have already been commissioned by the President and these commissions have been confirmed by the Senate.

"It is believed that the passage of the bill referred to is a recognition of the engineering profession by the federal government which is most gratifying. I feel that you will be glad to bring this action of the Government to the attention of the Board of the American Society of Civil Engineers, for their information.

"Very truly yours,

"R. L. FARIS,

"Acting Superintendent."

The Award of the John Fritz Medal for 1917

The John Fritz Medal for 1917, awarded to Dr. Henry M. Howe for his "investigations in metallurgy, especially in the metallography of iron and steel", was presented in the Auditorium of the United Engineering Society's Building, New York City, on Thursday evening, May 10th, 1917.

Mr. Ambrose Swasey, Chairman of the John Fritz Medal Board of Award, presided, and addresses were delivered by Dr. Rossiter W. Raymond, Secretary Emeritus of the American Institute of Mining Engineers, and Dr. Ira N. Hollis, President of the American Society of Mechanical Engineers.

Professor Albert Sauveur, Chairman of the Board at the time the award was made, presented the medal, and the ceremonies concluded with the response of Dr. Howe.

LIST OF RECIPIENTS OF THE JOHN FRITZ MEDAL.

John Fritz	1902
Lord Kelvin	1905
George Westinghouse	1906
Alexander Graham Bell	1907
Thomas Alva Edison	1908
Charles Talbot Porter	1909
Alfred Noble	1910
Sir William Henry White	1911
Robert Woolston Hunt	1912
John Edson Sweet	1914
James Douglas	1915
Elihu Thomson	1916
Henry Marion Howe	1917

Report of the Committee on Special Committees Presented to the Board of Direction, June 11th, 1917, and Ordered Printed for the Information of Members

30 CHURCH STREET, NEW YORK,
June 5, 1917.

THE BOARD OF DIRECTION,
AMERICAN SOCIETY OF CIVIL ENGINEERS,
New York, N. Y.

GENTLEMEN.—At your last meeting, April 17th, 1917, you referred to the Committee on Special Committees two matters for investigation and report, regarding which we beg to submit the following report:

1.—Request for the Appointment of a New Special Committee on Concrete and Reinforced Concrete.

We have given careful consideration to the possible advantages to the Society of continuing the Special Committee on Concrete and Reinforced Concrete or of the appointment of a new Special Committee. Since the date of the recent report there is nothing new in the art which would warrant continuing the old, or appointing a new, Special Committee to continue the work beyond the scope of the final report which was presented in January. There is no warrant under our Constitution for maintaining a Special Committee perpetually to watch developments in any particular branch of the art of engineering.

Our recommendation is that no such Special Committee should be continued nor new Special Committee appointed. In the future, when

there shall have been some further advance in the use of concrete and reinforced concrete sufficient to warrant a further study and report on the subject, the appointment of a Special Committee can be considered on its merits.

2.—Request for the Appointment of a Special Committee of Five Members to Investigate and Report on the Conditions and Opportunities for American Engineers in Russia and in South America.

Article VI, Section 12, of the Constitution of the Society, provides that, "Special committees to report upon engineering subjects shall be authorized, except as further provided in this paragraph", etc. In our opinion, the matter of investigating the conditions and opportunities for American Engineers in Russia and in South America does not come within the scope of the meaning of "engineering subjects", and we, therefore, recommend against the appointment of such a Committee as is requested.

In submitting this report, we beg to state that while it is not signed by Mr. W. L. Darling, a member of the Committee on Special Committees, the recommendations herein made are fully concurred in by him. The absence of Mr. Darling's signature hereto is due to his leaving this country on government business prior to the report being written.

Committee on Special Committees.
J. V. DAVIES, *Chairman.*
GEO. W. TILLSON.

Contingent Fees

The following letter, relating to contingent fees, was presented to the Board of Direction at its meeting on June 11th, 1917, and was ordered printed for the information of the membership.

"NEW ORLEANS, LA.
"May 29th, 1917.

"TO THE BOARD OF DIRECTION,
AMERICAN SOCIETY OF CIVIL ENGINEERS,
220 WEST 57TH ST., NEW YORK.

"GENTLEMEN.—Referring to the correspondence published in the May number of *Proceedings** in regard to the acceptance by engineers of a contingent fee, it would appear to the writer that this subject is of sufficient importance to justify further discussion, extending into fields other than service as an expert witness in Court. It may first, however, be proper to express a hearty agreement with the reply given by Mr. Clemens Herschel to the inquiry of Mr. J. H. Quinton as regards testimony given in Court. Our profession is in no position

* *Proceedings*, Am. Soc. C. E., May, 1917, p. 332.

to regulate the operation of our law Courts, but we can at least apply to our service in connection with those Courts the ideals which we stand for in our other relations. In most trials in American Courts, the attorneys are not serving as a semi-official part of the Court, but as violent partisans, striving, by every means which their code of ethics will permit, to secure an advantage for their particular client. The net result is that the side which employs the preponderance of ability secures the preponderance of advantage, especially in trials by jury and in the lower Courts. To offset, as much as may be practicable, this generally admitted pernicious practice, it would appear to be the duty of a member of our profession, when employed as an expert witness, to adopt the attitude that he is employed by the Court (though of necessity in some States paid by the client) to assist in arriving at the truth. Owing to the fact that this attitude has not been fixed by precedent long followed by all professions, the engineer's position might well be stated to the client or his attorney before accepting employment.

"The writer has occasionally found practicing attorneys who have so departed from the usual custom of their profession that, in the handling of some peculiar case, the technical details of which are not fully covered by their own experience, they will call in the services of some specialist in that line for assistance in the preparation of their brief, and more especially in the preparation of questions to be asked the witnesses. In such a case, the writer can see no objection to the engineer taking a partisan attitude, for he is then an assistant to the attorney and not to the Court. The question of a contingent fee under such circumstances might be judged more from a business than from an ethical standpoint.

"The average man who maintains a general engineering office is probably more often requested to render services for a contingent fee in connection with a new project requiring an examination and a report than for service as an expert witness in Court. This office has had an average of not less than four such cases offered per year for the past several years. Such proposals have been uniformly rejected for the following reasons:

"1.—Handling of engineering work on a contingent fee is considered to be poor business policy.

"2.—It is considered to be unethical.

"Considering these points in the order named, established custom prevents an engineer from charging a fee commensurate with the risks involved. Judged by the rule of averages, he would have to provide for a probable failure of the project (and consequent failure to collect his fee) in at least one-half the cases so handled. On all lost cases, he would be out, not only his own time, but also for expenses incurred. In many instances, these expenses run up into an amount which the average engineer is in no position to lose.

"As regards the ethical questions involved, assume that the engineer is employed to make an examination, survey, and report on a projected interurban railway. Acting in the capacity in which he is employed, he is not only charged with the duty of securing such information

as is desired by the promoters of the scheme, but, in a certain broad sense, he is the representative of all parties concerned. These parties are numerous, such as the promoters themselves, the future stockholders, and bondholders, the future patrons of the road, and all interests which might be affected by its construction and operation. Serving in such a capacity, the work of an engineer is, or should be, exceptionally judicial in its character. The engineer may be able to satisfy himself that his report will be in no way colored, modified, or exaggerated by the fact that the payment of his fee is contingent on the successful financing of the project, but human nature is weak, and even though he may satisfy his own conscience in the matter, the engineer may not always be able to satisfy the public, and more especially the canny investor, that he has been entirely unbiased in his presentation of the case, when it is known that it has been accepted on a contingent fee.

"The foregoing applies more particularly to the preparation of original plans and reports. After a project has been financed, no objection can be seen to the engineer accepting an administrative or other position in the concern, accepting in return payment or part payment in stock or profits, this again being more a matter of business than of ethics. It is obvious, however, that, in connecting himself with a project as part owner, the engineer may not consistently clothe himself with such broad powers over contractors who may be employed as is customarily done by engineers who take the position that they are unbiased arbitrators as between the owners and contractors. An attempt to lay down rules, however, for all such combinations of circumstances, would lead logically to a regulation in minute detail of all the acts of the individual, a procedure which is neither possible nor desirable. The only safe rule to apply is the Golden Rule, on which are based the generally accepted code of ethics of our profession.

"Respectfully submitted

"ARTHUR M. SHAW,

"Member, Am. Soc. C. E."

Proposed Revision of the Constitution

Referring to the note on page 315* with regard to the action of the Board in the matter of the proposed revision of the Constitution, the following facts should be noted:

The Report received by the Board was from a Committee consisting of M. T. Endicott, Chairman, J. A. Ockerson, George F. Swain, Hunter McDonald, J. V. Davies, H. S. Crocker, and Chas. Warren Hunt, and was not unanimous, the only difference of opinion in the Committee being as to whether the Secretary of the Society should be a member of the Board of Direction or not, Messrs. Endicott, Ockerson, McDonald, and Davies, holding that he should not be, and Messrs. Swain, Crocker, and Hunt, that he should be.

It should also be stated that the proposed By-laws are not intended to be passed upon by vote of the Society. They are published with the

* *Proceedings*, Am. Soc. C. E., May, 1917.

proposed Constitution, which is to be voted upon, for the information of the membership. Should a new Constitution be adopted, these By-laws will be subject to change or amplification by the Board of Direction.

Bill for the Licensing of Engineers Adopted by the State Legislature of Florida

This bill was adopted by the Senate of the State Legislature of Florida on May 17th, 1917, and by the House on the following day. A copy of the bill was forwarded to the Society by A. F. Harley, M. Am. Soc. C. E., and is here printed for the information of the membership.

AN ACT PROVIDING FOR THE CREATION AND ESTABLISHMENT OF A FLORIDA STATE BOARD OF ENGINEERING EXAMINERS, GRANTING CERTAIN POWERS TO AND PRESCRIBING THE DUTIES OF SAID BOARD; PROVIDING FOR THE EXAMINATION AND REGISTRATION OF PROFESSIONAL ENGINEERS, REGULATING THE PRACTICE OF ENGINEERING IN THE STATE OF FLORIDA, AND PROVIDING PENALTIES FOR THE VIOLATION OF THIS ACT.

Be It Enacted by the Legislature of the State of Florida:

SECTION 1. Definitions. As used in this Act:

(a) The "Board" means the State Board of Engineering Examiners provided for by this Act.

(b) A person practices "Professional Engineering" within the meaning of this Act who practices any branch of the profession of engineering other than military engineering. The practice of said profession embraces the design and the supervision of the construction of public and private utilities, such as railroads, bridges, highways, roads, canals, harbors, river improvements, lighthouses, wet docks, dry docks, ships, barges, dredges, cranes, floating docks and other floating property, the design and the supervision of the construction of steam engines, turbines, internal combustion engines and other mechanical structures, electrical machinery and apparatus, and of works for the development, transmission or application of power, the design and the supervision of mining operations and of processes and apparatus for carrying out such operations, and the design and the supervision of the construction of municipal works, irrigation works, water supply works, sewerage works, drainage works, industrial works, sanitary works, hydraulic works and structural works and other public or private utilities or works which require for their design or the supervision of their construction such experience and technical knowledge as are required in Section 8 of this Act for admission to examination. The execution as a contractor of work designed by a professional engineer or the supervision of the construction of such work as a foreman or superintendent for such a contractor shall not be deemed to be the practice of professional engineering within the meaning of this Act; nor shall this Act apply to a person acting as a public officer employed by the State, a County or a Municipality of this State on work where the estimated cost of the same is $1 000.00 or less.

(c) "Professional Engineer" means any person who practices professional engineering.

Sec. 2. After the first day of January, nineteen hundred and eighteen, no person shall practice professional engineering without having first been duly and regularly registered by the Board as a professional engineer as required by this Act, nor shall any person practice professional engineering whose authority to practice is revoked by the Board; and after the first day of January, nineteen hundred and eighteen, every map, plan and drawing required by law to be certified or approved by a professional engineer shall be certified or approved by a professional engineer duly and regularly registered by the Board as a professional engineer as required by this Act and shall bear the date and the number of the certificate of registration of such professional engineer.

Sec. 3. There shall be a State Board of Engineering Examiners consisting of five members to be appointed by the Governor, three of whom shall be civil engineers, one a mining or electrical engineer, and the other one a mechanical engineer or naval architect. Of the members of the Board first appointed hereunder two shall hold office for a term of two years ending the first day of July, nineteen hundred and nineteen. Two shall hold office for a term of three years ending the first day of July, nineteen hundred and twenty. One shall hold office for a term of four years ending the first day of July, nineteen hundred and twenty-one. The term of office of each member so appointed shall begin on the first day of July, nineteen hundred and seventeen. Upon the expiration of each of such terms the term of office of each member thereafter appointed shall be four years. Each member shall hold over after the expiration of his term until his successor shall be duly appointed and qualified. The Governor may remove any member of the Board for misconduct, incapacity or neglect of duty. Vacancies in the Board caused by death, resignation or removal from office shall be filled by appointment by the Governor for the unexpired term. Each member of the Board shall be a professional engineer of at least ten years active experience and of recognized good standing in his profession and shall be at least thirty-five years of age and shall have been a resident of this state for at least three years immediately preceding his appointment. Each member of said Board, except the members first appointed hereunder, shall also be registered as a professional engineer under this Act, and be also a member in good standing of a recognized engineering society. The members of the Board shall serve without compensation except traveling and other necessary expenses.

Sec. 4. Every member of the Board shall receive a certificate of his appointment from the Governor and before beginning his term of office shall file with the Secretary of State his written oath for the faithful discharge of his official duty. Each member of the Board first appointed hereunder shall receive a certificate of registration under this Act from said Board. The Board shall adopt and have an official seal. The Board may make all by-laws and rules not inconsistent with law needed in performing its duties; but no by-law or rule by which more than a majority vote is required for any specified action

by the Board shall be amended, suspended or repealed by a smaller vote than that required for action thereunder.

SEC. 5. The Board shall biennially elect from its members a president, a vice-president and a secretary who shall also be treasurer, for the ensuing biennial term. The Secretary shall give a bond in such amount and with such sureties as may be approved by the Board conditioned upon the faithful performance of his duties and for the accounting for, and payment of, all moneys received by him. The Secretary shall keep on file a record of all certificates of registration issued, and he shall receive and account for all fees derived from the operation of this Act.

The Board shall hold at least two regular meetings in each year. Special meetings may be called in such manner as the by-laws of the Board may provide. Notice of all meetings shall be given in such manner as the by-laws of the Board may provide. At all meetings a majority of the Board shall constitute a quorum.

SEC. 6. The Board shall have power to employ, during its pleasure, such clerks and other employees and to rent such offices as may be necessary for the proper performance by it of its duties as in this Act prescribed.

SEC. 7. The expenses of the Board and of the examinations held by the Board and of any other matter in connection with the provisions of this Act shall be paid from the registration fees collected as herein provided and not otherwise and in no case shall any of such expenses be paid by the State of Florida nor be charged against said State.

The members of the Board shall be entitled to reimbursement for their traveling and hotel expenses incurred in pursuance of their duties.

Any surplus funds remaining in the hands of the Board or of its Secretary and Treasurer shall be subject from time to time to the further provisions of the Legislature.

SEC. 8. The Board shall admit to examination any candidate who pays a fee of Fifteen Dollars ($15.00) and submits evidence, verified by oath and satisfactory to the Board, that he

(a) Is more than twenty-one years of age,
(b) Is of good character, and
(c) Has been engaged upon engineering work for at least six years and during that period has had charge of engineering work, as principal or assistant, for at least one year;
(d) Or, in lieu of requirement (c) specified above, is a graduate from an engineering school of recognized good reputation and has been engaged upon engineering work for at least four years and during that period has had charge of engineering work, as principal or assistant, for at least one year.

SEC. 9. Examinations for registration shall be held at regular or special meetings of the Board at such times and at such places within the State in each year as the Board shall determine. The scope of the examinations and the methods of procedure shall be prescribed by the Board. The examination may be either oral or partly oral and partly written. As soon as practicable after the close of each examination

the members of the Board who shall have conducted such examination shall make and sign and file with the Secretary a certificate stating the action of the Board upon the application of each candidate, whereupon the Secretary of the Board shall notify each candidate of the result of his examination.

Sec. 10. Upon receipt of an additional fee of Ten Dollars ($10.00) the Board shall issue to any applicant who has been certified as having passed the examination conducted by the Board a certificate of registration signed by the President and Secretary of the Board under the seal of the Board, whereupon each applicant shall be authorized to practice professional engineering as defined by this Act.

The Board shall from time to time examine the requirements for the registration of professional engineers in other States, territories and countries and shall record those in which, in the judgment of the Board, standards not lower than those provided by this Act are maintained. The Secretary of the Board, upon the presentation to him by any person of satisfactory evidence that such person holds a certificate of registration issued to such person by proper authority in any such State, territory or country so recorded and upon receipt by him of a fee of Twenty-five dollars ($25.00), shall issue to such person a certificate of registration under this Act, signed by the President and Secretary under the seal of the Board, whereupon the person to whom such certificate is issued shall be entitled to all the rights and privileges conferred by a certificate issued after examination by the Board.

Sec. 11. The Board shall at any time on or before the thirty-first day of December, nineteen hundred and seventeen, issue a certificate of registration signed by the President and the Secretary of the Board under the seal of the Board, upon due application therefor and the payment of a fee of Twenty-five Dollars ($25.00), to any professional engineer who shall submit evidence, verified by oath and satisfactory to the Board, that he is of good character and has been a resident of the State of Florida for at least one year immediately preceding the date of his application, and has practiced professional engineering for at least ten years preceding the date of his application and during that period has had charge of engineering work as principal or assistant for at least two years. After the thirty-first day of December, nineteen hundred and seventeen, the Board shall issue certificates of registration only as herein provided.

Sec. 12. All certificates of registration issued by the Board and its Secretary shall be in such form as the by-laws of the Board may prescribe. Before any certificate of registration is issued by the Board and its Secretary, it shall be numbered and recorded in a book kept for that purpose with the Secretary and the number of the certificate shall be noted on the certificate. Such record shall be open to public inspection and in all actions or proceedings in any court such record or a transcript of any part thereof certified by the Secretary of the Board under the seal of the Board to be a true copy shall be entitled to admission in evidence.

The Board shall have power at any time to inquire into the identity of any person claiming to be a registered professional engineer and after due service of a notice in writing, require him to prove to the satisfaction of the Board, that he is the person authorized to practice

professional engineering under the certificate of registration by virtue of which he claims the privilege of this Act.

When the Board finds that a person claiming to be a professional engineer registered under this Act is not in fact the person to whom the certificate of registration was issued, it shall reduce its findings to writing and file them in its office. Such findings shall be prima facie evidence that the person mentioned therein is falsely impersonating a professional engineer of a like or different name.

The Board may revoke a certificate of registration of a professional engineer for fraud or deceit in the securing of his certificate or for the conviction of crime. Proceedings for the revocation of a certificate of registration shall be begun by filing with the Board a written charge or charges against the accused. These charges may be preferred by any person or corporation or the Board may on its own motion direct its Secretary to prefer such charges. When charges are preferred against a professional engineer, registered under the provisions of this Act, the Board shall designate not less than three of its number as a committee to hear and determine said charges. A time and place for the hearing shall be fixed by said committee and a copy of the charges, together with a notice of the time and place for the hearing, shall be served upon the accused or his counsel at least ten days before the date fixed for said hearing. Where personal service or service upon counsel cannot be effected and such fact is certified on oath by any person duly authorized to make legal service, the Board shall cause to be published for at least five times, the first publication to be at least thirty days prior to the hearing, in two newspapers published in the section of the State in which the accused was last known to practice, a notice to the effect that at a definite time and place a hearing will be held on the charges against the accused upon an application to revoke his certificate. At said hearing the accused shall have the right to cross-examine witnesses against him and produce witnesses in his defense and to appear personally or by counsel. The said committee shall make a written report of its findings and recommendations and shall forthwith submit the same to the Board. If the Board shall sustain the findings of the committee and it is the decision of the Board that said certificate should be revoked the Board shall thereupon give written notice to the said professional engineer against whom the charges are preferred of its intention so to do, whereupon the said professional engineer against whom said charges have been preferred shall have the right within sixty days to appeal to any court in equity or law, having proper jurisdiction, against the action of the said Board and the action of the Board shall be subject to review and decision of said court, or of a higher court, if appeal is taken. In the event that the said professional engineer against whom charges have been preferred shall not within sixty days appeal from the decision of the Board in the manner above provided, the Board shall have the power and authority to thereupon, or as soon thereafter as practicable, annul and revoke the said certificate of registration. The action of the Board shall be recorded in the same manner as certificates of registration are recorded and the name of the person whose certificate of registration is so revoked shall be stricken from the list of registered professional engineers and he

shall be disqualified from practicing as a professional engineer in the State of Florida.

The Board may re-issue a certificate of registration to any person whose certificate has been revoked, but only after the expiration of one year from the date of such revocation, for reasons which the Board shall determine to be satisfactory.

SEC. 13. Every certified professional engineer so registered under this Act who desires to continue the practice of his profession shall annually pay to the Secretary of the Board a fee of Five Dollars ($5.00) on or before a date to be fixed by the Board, for which fee a renewal certificate of registration for the current year shall be issued.

SEC. 14. It shall be unlawful for any professional engineer to participate in or derive any profit from the subject matter of his professional employment, either in the matter of construction work or materials.

SEC. 15. Every unrevoked certificate and endorsement of registry made as provided in this Act shall be presumptive evidence in all courts and places that the person named therein is legally registered.

SEC. 16. The provisions of this Act shall apply to every corporation, domestic or foreign, engaged in the business of professional engineering within the State of Florida, except that certificates of registration issued hereunder shall be held by one or more of its officers or employees instead of by such corporation.

SEC. 17. The Board shall, during the month of April in each year, certify and publish a complete list of registered professional engineers with their business addresses in a newspaper published in the State of Florida.

SEC. 18. Any person who, not being then legally authorized to practice professional engineering within this State according to the provisions of this Act and so registered according to law, shall practice, or attempt or advertise to practice, or hold himself out as authorized to practice professional engineering, or shall use in connection with his name, or otherwise assume, use or advertise any title or designation tending to convey the impression that he is a professional engineer, and any person who shall buy, sell or fraudulently obtain any certificate of registration or who shall aid or abet such buying, selling or fraudulently obtaining or who shall practice, or attempt or advertise to practice or hold himself out as authorized to practice professional engineering under cover of any certificate obtained or issued fraudulently or unlawfully or under fraudulent representations or wilful misstatement of fact in a material regard and any person who shall practice, or attempt or advertise to practice, or hold himself out as authorized to practice professional engineering under a false or assumed name or who shall falsely impersonate any professional engineer or former professional engineer of a like or different name or who shall violate any of the provisions of this Act, shall be guilty of a misdemeanor, and upon conviction thereof shall be punished according to law, and in addition his certificate of registration shall be automatically revoked.

SEC. 19. This Act shall not apply to any professional engineer working for the United States Government; nor to any professional engineer employed as an assistant to a professional engineer registered under this Act; nor to any professional engineer coming from without this State and employed therein until a reasonable time, as prescribed by the rules of the Board, shall have elapsed to permit the registration of such person under this Act, provided that before practicing within this State he shall have applied for the issuance to him of a certificate of registration and shall have paid the fee prescribed in this Act for admission to examination.

SEC. 20. This Act shall not apply to any architect registered by the State of Florida under the provisions of the Act creating the State Board of Architecture nor to any of the provisions of said Act.

SEC. 21. All laws or parts of laws in conflict with the provisions of this Act be and the same are hereby repealed.

SEC. 22. This Act shall take effect immediately upon its passage and approval by the Governor.

Report of a Joint Committee on Organization of an American Engineers Standards Committee

to the Governing Boards of the American Society of Civil Engineers, American Institute of Mining Engineers, American Society of Mechanical Engineers, American Institute of Electrical Engineers, and the American Society for Testing Materials.*

PREAMBLE.

At the present time many bodies are engaged in the formulation of standards. There is no uniformity in the rules for such procedure in the different organizations; in some cases the committees engaged in the work are not fully representative, and in a considerable proportion of cases they do not consult all the allied interests. The present custom results in a considerable duplication of work, and there are in some fields several "standards" proposed for the same thing that differ from each other only slightly and that often in unimportant details. It is very much more difficult to obtain agreement between the proposers of overlapping standards after they have been published than it would be to get the proposers to agree before they had committed themselves publicly.

The American Society of Civil Engineers, the American Institute of Mining Engineers, the American Society of Mechanical Engineers, the American Institute of Electrical Engineers, and the American Society for Testing Materials, having appointed this committee to consider the advisability of co-operating in Engineering Standardization, we recommend that these societies form a permanent organization by

* This report has not yet been presented to the Board of Direction, but is printed here for the information of the membership. The members of the American Society of Civil Engineers on this Committee were: W. H. Burr, J. H. Gregory, and H. N. Latey.

appointing a committee, as proposed in the following constitution and rules of procedure, to carry on this important work.

CONSTITUTION.

1.—The name of this committee shall be the American Engineering Standards Committee, hereinafter referred to as the Main Committee.

2.—The object of this committee is to unify and simplify the methods of arriving at engineering standards, to insure co-operation between the different societies, and to prevent duplication of work.

3.—This committee shall be composed of three representatives from each of the societies mentioned in the preamble and from such other societies as may be added later, the appointments to be made by the governing bodies of the respective societies.

4.—At its first meeting the committee shall divide itself into three groups, to serve one, two, and three years, one representative of each society to be in each group. Each year after the first, each society shall appoint one representative to serve for three years. Representatives may be reappointed. A vacancy shall be filled by the society to which the retiring representative belonged.

5.—*Officers.*—The committee shall elect annually a chairman and vice-chairman from its own membership. Officers so elected shall serve for one year, or until their successors are elected. These officers shall not serve for more than three consecutive terms. Vacancies may be filled by election at any regular or special meeting, providing notice has been given in the call for the meeting.

6.—The committee shall elect from its own members an Executive Committee of not less than five, and may delegate to it any of its powers, except that of approving "Recommended Practices" and "American Standards." (See Paragraph 13.)

7.—*The Executive Committee* shall engage a secretary who shall not be a member of the committee. This engagement shall be made annually within three months of the annual election.

8.—*Meetings.*—The committee shall hold at least four regular meetings each year on specified dates. It may hold additional meetings at any time at the call of the chairman on not less than ten days' notice.

9.—*Quorum.*—A quorum of the Main Committee for the transaction of business shall consist of at least two-thirds of the membership of the committee. A quorum of the Executive Committee shall be not less than a majority.

10.—*Duties.*—The chief duties of the Main Committee shall be:

a.—To receive and pass upon recommendations for standards submitted by Sectional Committees, but not to deal with the details of any particular standard;

b.—To formulate rules under which the Sectional Committees shall be constituted, organized, and conduct their work;

c.—To maintain a central office in the United Engineering Societies' Building in New York with a paid full-time secretary, which office shall also serve as a bureau of information.

11.—A Sectional Committee is a committee whose personnel or composition has been approved by the Main Committee as being sufficiently comprehensive and authoritative to prepare a particular standard, or group of standards, for submission to the Main Committee. It may be: the standing committee on Standards of an individual "co-operating society"; a special committee appointed by a "co-operating society" for the consideration of a special standard, or group of standards, within the obvious field of that society; a joint committee appointed by two or more societies; or as in such a case as that of the American Society for Testing Materials, the society itself.

12.—The term "Co-operating Society" as here used refers, not only to any one of the societies represented on the Main Committee, but also to any other society, a Government Technical Body, such as the Bureau of Standards, or an Army or Navy Construction Board, a Public Service Commission, or any other significant body vitally concerned with engineering standards. It is understood, however, that, as far as relates to the standards proposed, all co-operating societies shall abide by the rules laid down by the Main Committee.

13.—Any proposed standard submitted to, and approved by, the Main Committee shall be known as "Recommended Practice", or when, in the opinion of this committee it has proved its suitability, it shall be recommended as an "American Standard."

14.—The approval as "Recommended Practice" of any standard submitted to the Main Committee shall require the affirmative vote of three-fourths of the committee; the advance of status from "Recommended Practice" to "American Standard" shall require an affirmative vote of 90% of the Main Committee. Such votes may be by letter-ballot. Letter-ballots may be ordered at any regular or special meeting of the Main Committee.

15.—*Expenses.*—As soon as possible after its organization the Main Committee shall make an estimate of its expenses for the first year and submit it to the governing bodies of the societies represented on the Main Committee for their approval; such approval includes the pledge of each society for such share as may be agreed upon. After the first year the Main Committee shall present an annual budget to the governing bodies of the societies represented, not later than December 1st, each year.

Funds of the committee shall be in the custody of the secretary, who shall be placed under suitable bond, and shall be disbursed by him on vouchers signed by the chairman or vice-chairman.

16.—*Amendments* must be proposed in writing at least thirty days before the meeting of the Main Committee at which they are to be voted on. If passed by a three-fourths majority of those present, they shall be referred to the Governing Boards of all the societies represented in the Main Committee, and shall become operative only when all these Boards have given their approval.

Rules of Procedure.

1.—Each co-operating society shall notify the Main Committee of the names and affiliations of the members of, and the field covered by, any existing or proposed committee dealing with a standard of any

kind. It shall also send to the Main Committee not less than twenty copies of each standard which it has in force.

This information shall be assembled, classified, and kept up to date by the Secretary of the Main Committee, and shall be accessible at any time to each of the co-operating societies.

2.—Any proposal for a particular standard or group of standards shall be referred to the Main Committee, which shall designate an appropriate sectional committee to carry on the work.

3.—*Composition of Committees:*

(*a*) Sectional Committees dealing with standards of a commercial character (specifications, shop practices, etc.), shall be made up of representatives of producers, consumers, and general interests, no one of these interests to form a majority. A producer is a person, or the representative of a firm or corporation, directly concerned in the production of the commodity involved. A consumer is a person, or the representative of a firm or corporation, that uses the commodity involved, but is not directly concerned with its production. General interests include independent engineers, educators, and persons who are neither consumers nor producers, as defined above.

(*b*) Sectional Committees dealing with standards of a scientific or non-commercial character shall consist of persons specially qualified, without regard to their affiliations.

4.—*Sectional Committees* may consult specialists, or obtain any other evidence they may deem advisable.

5.—Every report of a Sectional Committee shall include a list of its members, with a statement of their business, professional, and technical society affiliations. The final vote shall be stated in detail, and attested by the signature, personal or authorized, of each member voting.

6.—Should a Sectional Committee be unable to agree on a report by a three-fourths majority, then the Main Committee may request the appointing society or societies to discharge it and appoint a new one; or the Main Committee may, with the approval of the societies interested, appoint a special committee to hear both sides and present recommendations.

7.—Co-operating societies shall not, in their publications, use the terms "Recommended Practice" or "American Standard" except in connection with standards that have received the approval of the Main Committee.

8.—"Recommended Practices" and "American Standards" approved by the Main Committee shall be copyrighted, and shall be printed in the official publications of the society or societies proposing them; they shall not be released prior to such publication.

9.—*Amendments* to the Rules of Procedure may be proposed at any meeting of the Main Committee, and, if so ordered, shall be sent in full to all members with the notice of a subsequent meeting at which they may be voted on. A three-fourths affirmative vote of those present will be required to pass an amendment.

C. A. Adams,
Chairman.

June 19th, 1917.

ANNOUNCEMENTS

The House of the Society is open from 9 A. M. to 10 P. M., every day, except Sundays, Fourth of July, Thanksgiving Day, and Christmas Day.

FUTURE MEETINGS

September 5th, 1917.—8.30 P. M.—A regular business meeting will be held, and a paper by Floyd A. Nagler, Jun. Am. Soc. C. E., entitled "Obstruction of Bridge Piers to the Flow of Water", will be presented for discussion.

This paper was printed in *Proceedings* for May, 1917.

September 19th, 1917.—8.30 P. M.—At this meeting a paper by Ray W. Berdeau, Jun. Am. Soc. C. E., entitled, "The Three 15-Cubic Yard Dipper Dredges, *Gamboa, Paraiso,* and *Cascadas,* as Supplied and Used on the Panama Canal", will be presented for discussion.

This paper is printed in this number of *Proceedings.*

October 3d, 1917.—8.30 P. M.—This will be a regular business meeting. A paper by William Barclay Parsons, M. Am. Soc. C. E., entitled "The Cape Cod Canal", will be presented for discussion.

This paper is printed in this number of *Proceedings.*

SEARCHES IN THE LIBRARY

In January, 1902, the Secretary was authorized to make searches in the Library, upon request, and to charge therefor the actual cost to the Society for the extra work required. Since that time many searches have been made, and bibliographies and other information on special subjects furnished.

The resulting satisfaction, to the members who have made use of the resources of the Society in this manner, has been expressed frequently, and leaves little doubt that if it were generally known to the membership that such work would be undertaken, many would avail themselves of it.

The cost is trifling compared with the value of the time of an engineer who looks up such matters himself, and the work can be performed quite as well, and much more quickly, by persons familiar with the Library.

In asking that such work be undertaken, members should specify clearly the subject to be covered, and whether references to general books only are desired, or whether a complete bibliography, involving search through periodical literature, is desired.

It sometimes happens that references are found which are not readily accessible to the person for whom the search is made. In that case the material may be reproduced by photography, and this can be done for members at the cost of the work to the Society, which is

small. This method is particularly useful when there are drawings or figures in the text, which would be very expensive to reproduce by hand.

A list of 989 bibliographies made in the Library, giving the cost of each, was published in Vol. LXXX of *Transactions.*

Since October 1st, 1916, the Library of the American Society of Civil Engineers has ceased to exist, as such, having been merged with the Libraries of the Mining, Mechanical, and Electrical Engineers, and become a part of the Library of the United Engineering Society. There were 67 000 accessions, which were not duplicates, turned over to that Library.

Hereafter, therefore, requests for searches should be addressed to the Librarian, United Engineering Society, 29 West 39th Street, New York City.

PAPERS AND DISCUSSIONS

Members and others who take part in the oral discussions of the papers presented are urged to revise their remarks promptly. Written communications from those who cannot attend the meetings should be sent in at the earliest possible date after the issue of a paper in *Proceedings.*

All papers accepted by the Publication Committee are classified by the Committee with respect to their availability for discussion at meetings.

Papers which, from their general nature, appear to be of a character suitable for oral discussion, will be published as heretofore in *Proceedings,* and set down for presentation to a future meeting of the Society, and on these, oral discussions, as well as written communications, will be solicited.

All papers which do not come under this heading, that is to say, those which, from their mathematical or technical nature, in the opinion of the Committee, are not adapted to oral discussion, will not be scheduled for presentation to any meeting. Such papers will be published in *Proceedings* in the same manner as those which are to be presented at meetings, but written discussions only will be requested for subsequent publication in *Proceedings* and with the paper in the volumes of *Transactions.*

The Board of Direction has adopted rules for the preparation and presentation of papers, which will be found on page 429 of the August, 1913, *Proceedings.*

LOCAL ASSOCIATIONS OF MEMBERS OF THE AMERICAN SOCIETY OF CIVIL ENGINEERS

San Francisco Association, Organized 1905.

J. D. Galloway, President; E. T. Thurston, Secretary-Treasurer, 57 Post Street, San Francisco, Cal.

The San Francisco Association of Members of the American Society of Civil Engineers holds regular bi-monthly meetings, with banquet, and weekly informal luncheons. The former are held at 6 P. M., at the Palace Hotel, on the third Tuesday of February, April, June, August, October, and December, the last being the Annual Meeting of the Association.

Informal luncheons are held at 12.30 P. M., every Wednesday, and the place of meeting may be ascertained by communicating with the Secretary.

The by-laws of the Association provide for the extension of hospitality to any member of the Society who may be temporarily in San Francisco, and any such member will be gladly welcomed as a guest.

(Abstract of Minutes of Meetings)

April 17th, 1917.—The meeting was called to order at the Palace Hotel; President Galloway in the chair; E. T. Thurston, Secretary; and present, also, 87 members and guests.

President Galloway, for the Committee on Military Affairs, reported the successful progress of lecture courses by Army officers, and also in regard to enlistments in the Officers' Reserve Corps.

The Committee on Past-President Haehl's Recommendations for the good of the Association, through Mr. W. L. Huber, and the Committee on the Frickstad case, through Mr. W. J. Allan, reported matters at a standstill.

The Committee on Civil Service, through Mr. J. Newman, reported that the Committee was unable at this time to make a complete report, but Mr. Newman read a letter addressed to the Mayor and the Board of Supervisors of San Francisco, indicating the official attitude of the Association relative to the salaries of civil service employes of that city.

The Committee on Discussion of Society Proceedings, H. D. Dewell, Chairman, reported that the suggestion that special meetings in alternate months be held for that purpose, had been considered, and asked for an expression of opinion on the subject on the part of the meeting. The Committee was continued, and ordered to present a definite report at the next meeting.

The Committee appointed to oppose and prevent the passage of the proposed Architects' License Law before the State Legislature, through Mr. C. H. Snyder, presented a brief report of the Committee's trip to Sacramento and its temporary success in having the measure tabled in the Assembly Committee. Relative to this matter, the Secretary reported that the bill had been again taken up by the Assembly Committee and, after amendment, had been passed out of Committee with favorable recommendations. The Committee of the Association was continued and instructed to continue its efforts against the bill until the adjournment of the Legislature.

The Committee on Revision of Building Laws of San Francisco reported progress, through Mr. H. J. Brunnier.

The Secretary announced that a communication from the St. Louis Association proposing certain amendments to the Constitution of the Society had been discussed by the Board of Directors, which decided that action on this matter be delayed pending information as to the

report of the Committee of the Society on Amendments to the Constitution. He also announced the endorsement by the St. Louis Association of the suggestions that amendments to the Constitution be submitted separately for informal discussion and expression of opinion by the various Associations before the complete document is submitted for vote to the membership of the Society.

President Galloway reported that he had received a communication from the Professional Classes War Relief Council, Inc., requesting contributions for the fund for the relief of distressed families of engineers in Great Britain who had been forced out of employment by the war. He stated that the Board of Directors had suggested a donation of $100, but after discussion, on motion, duly seconded, the contribution of the Association was raised to $500.

Communications from the Navy Yard, Mare Island, relative to applications for examinations for commissions in the Civil Engineers Corps of the Navy, and from T. C. Desmond and Company, New York City, relative to enlistment of engineers in the proposed volunteer regiment to be organized under the command of Colonel Roosevelt for immediate service in France, were referred to the Committee on Military Affairs.

Mr. W. A. Cattell, Secretary of the International Engineering Congress, read an interesting statement relative to the origin, development, and finances of that Congress.

An interesting paper, consisting of personal reminiscences of work on the early railroads in the West, entitled "Fifty Years of Railroad Development in the Western United States", by Mr. William Hood, was presented by the author.

Adjourned.

June 19th, 1917.—The meeting was called to order at the Palace Hotel; President Galloway in the chair; E. T. Thurston, Secretary; and present, also, 48 members and guests.

Messrs. Beebee, Holly, and McWethy were appointed as the Entertainment Committee for the August meeting.

President Galloway, of the Military Affairs Committee, reported that the military lectures had been concluded because the Army officers were otherwise occupied. Reports in behalf of the Committees on the Frickstad matter, the Civil Service Laws, the discussion of Society Proceedings, and the Revision of Building Laws indicated little activity. The President reported the death of the Architects' License Bill. The organization of the Engineering Council of the National Engineering Societies was announced, and also the appointment of Mr. Galloway as one of the five members from the American Society of Civil Engineers.

The Secretary read a communication from Mr. L. B. Stillwell in appreciation of the contribution of the Association to the Professional Classes War Relief Council of Great Britain; reported having made the final payment on the new Western Pacific Railroad Corporation bond subscription, amounting to $580.25, finally establishing the ownership by the Association of one $800 bond, preferred stock to the value of $1 000, and common stock to the value of $1 900; and

announced the investment, on the authority of the Board of Directors, in $200 worth of Liberty Loan bonds.

As there remained on hand a surplus of $100 more than the anticipated expenditures for the year, this sum, on motion, duly seconded, was donated to the American Red Cross War Fund.

George W. Howson, Assoc. M. Am. Soc. C. E., delivered an address on "The Development of the Stanislaus River, with Special Reference to Strawberry Dam."

The subject of the address was discussed by Messrs. Galloway, Grunsky, O'Shaughnessy, and Vensano.

The report of the Committee on the Relations of Local Associations of the American Society of Civil Engineers to that Society, to other Engineering Organizations, and Engineers, and to the Public, as published in the *Proceedings* for May, 1917, was discussed, and disclosed a general approval of the report. On motion, duly seconded, it was unanimously resolved to endorse particularly the suggestion that local organizations be known as "Sections of the American Society of Civil Engineers" instead of "Associations of Members of the American Society of Civil Engineers."

Adjourned.

Colorado Association, Organized 1908.

Thomas W. Jaycox, President; L. R. Hinman, Secretary-Treasurer, 1400 West Colfax Avenue, Denver, Colo.

The meetings of the Colorado Association of Members of the American Society of Civil Engineers (Denver, Colo.) are held on the second Saturday of each month, except July and August. The hour and place of meeting are not fixed, but this information will be furnished on application to the Secretary. The meetings are usually preceded by an informal dinner. Members of the American Society of Civil Engineers will be welcomed at these meetings.

Weekly luncheons are held on Wednesdays at 12.30 P. M., at Daniel's and Fisher's.

Visiting members are urged to attend the meetings and luncheons.

(Abstract of Minutes of Meeting)

April 14th, 1917.—The meeting was called to order at the Denver Athletic Club; Vice-President Follansbee in the chair; L. R. Hinman, Secretary; and present, also, 14 members.

The minutes of the meeting of March 10th, 1917, were read and approved.

The Secretary read a letter from Thomas C. Desmond, Assoc. M. Am. Soc. C. E., requesting distribution of information relative to the Engineer Regiment in the proposed Roosevelt Division.

For the Committee on Co-operation, the Secretary outlined the work done thus far in the attempt of the various engineering societies of Colorado to effect a satisfactory joint organization, the efforts to date having been instigated by the Colorado Scientific Society.

For the Committee on Roads and Highways, Mr. A. E. Palen presented an abstract of the "Progress Report of the Special Com-

mittee on Materials for Road Construction and on Standards for Their Test and Use." In the discussion that followed, Messrs. Phelps and Comstock took part.

On motion, duly seconded, it was decided to hold a special meeting for the purpose of discussing the Standley Lake Dam, and also for the presentation of an abstract of the paper entitled "Multiple-Arch Dams on Rush Creek, California," by L. R. Jorgensen, M. Am. Soc. C. E., by the Committee on Dams.

Adjourned.

Atlanta Association, Organized 1912.

Paul H. Norcross, President; Thomas P. Branch, Secretary-Treasurer, Georgia School of Technology, Atlanta, Ga.

The Association holds its meetings at the University Club, Atlanta, Ga. Regular monthly luncheon meetings are held to which visiting members of the Society are always welcome.

Baltimore Association, Organized 1914.

Mason D. Pratt, President; Charles J. Tilden, Secretary-Treasurer, The Johns Hopkins University, Baltimore, Md.

Cleveland Association, Organized 1914.

W. J. Watson, President; George H. Tinker, Secretary-Treasurer, 516 Columbia Building, Cleveland, Ohio.

Detroit Association, Organized 1916.

T. A. Leisen, President; Clarence W. Hubbell, Secretary; 2334 Dime Bank Building, Detroit, Mich.

The regular meetings of the Association are held on the second Friday of December, April, and October, the last being the Annual Meeting.

District of Columbia Association, Organized 1916.

A. P. Davis, President; John C. Hoyt, Secretary-Treasurer, U. S. Geological Survey, Washington, D. C.

Duluth Association, Organized 1917.

F. E. House, President; Walter G. Zimmermann, Secretary, Wolvin Building, Duluth, Minn.

The regular meetings of the Association are held monthly. The time and place of meeting are not fixed, but this information will be furnished on application to the Secretary. The Annual Meeting is held on the third Monday of May.

Illinois Association, Organized 1916.

C. F. Loweth, President, Chicago, Ill.

The regular meetings of the Association are held on the second Monday of March, June, September, and December, the last being the Annual Meeting. The hour and place of meeting are not fixed, but this information will be furnished on application to the President.

Louisiana Association, Organized 1914.

W. B. Gregory, President; Charles W. Okey, Secretary, Tulane University, New Orleans, La.

The regular meetings of the Association are held at The Cabildo, New Orleans, La., on the first Monday of January, April, July, and October.

Nebraska Association, Organized 1917.

Frank T. Darrow, President; Homer V. Knouse, Secretary-Treasurer, 115 City Hall, Omaha, Nebr.

Regular meetings of the Association are held on the first Saturday of each month, except July and August, and at such places as may be appointed from time to time by the Executive Committee. The Annual Meeting is held in Lincoln, Nebr., on the second Friday in January.

It is probable that frequent luncheons will be held in Omaha, in addition to the monthly meetings, at which visiting members will be welcomed. The place of meeting may be ascertained by communicating with the Secretary.

(Abstract of Minutes of Meeting)

June 2d, 1917.—The meeting was called to order at 8.00 P. M., at the Paxton Hotel, Omaha, Nebr.; Vice-President Dobson in the chair; Homer V. Knouse, Secretary; and present, also, 7 members and 3 guests.

The minutes of the two preceding meetings were read and approved, as corrected.

A communication from E. T. Thurston, Secretary of the San Francisco Association, pertaining to the equitable apportionment of Vice-Presidents of the Society among the various Districts, was read by the Secretary.

The Secretary presented a letter from Mr. E. G. Haines relative to the work of the Society's Special Committee on the Bearing Value of Soils.

A communication from Chas. Warren Hunt, Secretary of the Society, requesting action by the Association on the relations of Local Associations with the Society and with other Local Associations and clubs, was read by the Secretary, and the Chairman called the attention of the Association to the advisability of immediate action.

As a basis for discussion of the subject, the Secretary read the Report of the Committee of the Board of Direction on the Relation of Local Associations of the American Society of Civil Engineers to that Society, to other Engineering Organizations and Engineers, and to the Public, as published in the *Proceedings* for May, 1917.*

After discussion by paragraphs, on motion, duly seconded, the report was endorsed, with the following changes: Under the sub-heading, "Local Societies", it was recommended that this paragraph, read as follows: "Local Sections should affiliate with existing Engineering Societies", and that the remainder of the paragraph be omitted.

* Pages 327–330.

On motion, duly seconded, it was decided as desirable that District No. 10 should have representation in the office of Vice-President of the Society, and that it be suggested to the Nominating Committee to nominate Mr. H. S. Crocker as a candidate for that office.

On motion, duly seconded, the Chairman was authorized to appoint a Committee of the Association to co-operate with the Society's Special Committee on the Bearing Value of Soils, and collect data on this subject. Messrs. McClintock, Mickey, and Arend were subsequently appointed as such a Committee.

The following resolution, to be presented to the Hon. Keith Neville, Governor of Nebraska, was adopted:

"*Whereas,* the five American Engineering Societies proffered the United States Government during 1916 their services in making an inventory of the industrial plants and resources of the country, which has been accomplished in a creditable manner, and

"*Whereas,* the country is now in a state of war and the need of further industrial inventory or of other assistance is probable; therefore be it

"*Resolved,* that the Nebraska Association of Members of the American Society of Civil Engineers at the regular meeting at Omaha, Nebraska, on June the second, nineteen hundred and seventeen, tender their services, collectively or individually, to the State for such duties as your Excellency may see fit."

Adjourned.

Northwestern Association, Organized 1914.

George L. Wilson, President; Ralph D. Thomas, Secretary, 508 South First Street, Minneapolis, Minn.

Philadelphia Association, Organized 1913.

Samuel T. Wagner, President; C. W. Thorn, Secretary, 1313 South Broad Street, Philadelphia, Pa.

The regular meetings of the Association are held at the Engineers' Club of Philadelphia, 1317 Spruce Street, on the first Monday in January, April, and October, the last being the Annual Meeting.

Portland, Ore., Association, Organized 1913.

J. P. Newell, President; J. A. Currey, Secretary, 194 North 13th Street, Portland, Ore.

St. Louis Association, Organized 1914.

J. A. Ockerson, President; Gurdon G. Black, Secretary-Treasurer, 34 East Grand Avenue, St. Louis, Mo.

The meetings of the Association are held at the Engineers' Club Auditorium. The Annual Meeting is held on the fourth Monday in November. The time of other meetings is not fixed, but this information will be furnished on application to the Secretary.

San Diego Association, Organized 1915.

W. J. Gough, President; J. R. Comly, Secretary-Treasurer, 4105 Falcon Street, San Diego, Cal.

Seattle Association, Organized 1913.

Joseph Jacobs, President; Carl H. Reeves, Secretary-Treasurer, 444 Henry Building, Seattle, Wash.

The regular meetings of the Association are held at 12.15 P. M., on the last Monday of each month, at The Frye Hotel.

(Abstract of Minutes of Meetings)

April 30th, 1917.—The meeting was called to order at 12.15 P. M., at The Frye Hotel; President Jacobs in the chair; Carl H. Reeves, Secretary; and present, also, 18 members and guests.

The minutes of the meeting of March 29th, 1917, were read and approved.

The resignation of Mr. L. R. Hjorth as a member of the Association was presented and accepted.

A communication from the Secretary of the National Conference on City Planning, *in re* the Annual Conference to be held in Kansas City, Mo., on May 7th-8th, 1917, was read, and the President stated that he would be glad to give proper credentials as Delegate to any member of the Association who could attend the Conference.

Mr. John L. Hall, President of The Associated Engineering Societies of Seattle and a member of the Executive Committee of the Central Council for Patriotic Service, presented an outline of the proposed work of the Council.

Mr. C. N. Kast, Field Engineer with the Interstate Commerce Commission, on railroad valuation, addressed the meeting briefly relative to the methods and objects of that work.

A letter from Mr. Thomas C. Desmond, of New York City, relative to the Roosevelt Division for service in France, was read, and the application blanks furnished by Mr. Desmond were distributed to those present. No action was taken by the Association on the subject-matter of the letter.

Adjourned.

May 28th, 1917.—The meeting was called to order at The Frye Hotel; President Jacobs in the chair; Carl H. Reeves, Secretary; and present, also, 23 members and guests.

The minutes of the meeting of April 30th, 1917, were read and approved.

The resignation of Mr. Roy E. Smith was accepted.

A letter from Chas. Warren Hunt, Secretary of the Society, relating to the report of a committee, appointed by the Board of Direction, on the Relations of Local Associations of the American Society of Civil Engineers to that Society, to other Engineering Organizations, and Engineers, and to the Public, was read and referred to the Committee on Relations with the Parent Society.

Mr. A. H. Fuller presented a brief outline of the work and objects of the National Research Council, and stated that this work was to be done in connection with the Council for the National Defense.

On motion, duly seconded, it was ordered that all members of the Association who have become affiliated with any of the arms of the service of the Government in connection with the prosecution

of the war, and while in actual service, be excused from the payment of dues.

Maj. E. J. Dent, U. S. A., spoke on matters in relation to the work of the Engineer Corps of the Army.

W. C. Weeks, M. Am. Soc. C. E., of Union Bay, B. C., Canada, spoke on labor and other conditions in British Columbia.

Adjourned.

June 25th, 1917.—The meeting was called to order at The Frye Hotel; President Jacobs in the chair; Carl H. Reeves, Secretary; and present, also, 24 members and guests.

The minutes of the meeting of May 28th, 1917, were read and approved.

The resignation of Mr. W. D. Shannon was accepted.

The Committee on Relations with the Parent Society presented an oral report on the matter affecting Local Associations, as presented in the *Proceedings* for May, 1917. The final report of this Committee was called for the July meeting, prior to which time the Secretary-Treasurer was instructed to have a copy of the report, as well as the Committee's previous report on similar matters, in the hands of each member.

The President called attention to the proposed revised Constitution of the Society, as printed in the *Proceedings* for May, 1917, and appointed Mr. A. H. Dimock to review the matter at the July meeting, if time should permit, or at a future meeting.

Adjourned.

July 30th, 1917.—The meeting was called to order at The Frye Hotel; President Jacobs in the chair; O. P. M. Goss, acting as Secretary; and present, also, 15 members and guests.

The minutes of the meeting of June 25th, 1917, were read and approved.

Mr. A. H. Fuller discussed the report of the Committee on Relations with the Parent Society. After various phases of the report had been discussed by Messrs. Dimock, Jacobs, Rathbun, Hall, and Howes, it was decided, on motion, duly seconded, that the Committee should modify the report by giving reasons for the adoption of various paragraphs. On motion, duly seconded, it was decided to amend the first paragraph to express the idea that the business of the Annual Meeting should be placed in the hands of a duly elected delegate only.

Mr. A. H. Dimock reviewed the proposed new Constitution of the Society.

Adjourned.

Southern California Association, Organized 1914.

H. Hawgood, President; Wilkie Woodard, Secretary, 435 Consolidated Realty Building, Los Angeles, Cal.

The Southern California Association of Members of the American Society of Civil Engineers (Los Angeles, Cal.) holds regular bi-monthly meetings, with banquet, at Hotel Clark, on the second Wednesday of

February, April, June, August, October, and December, the last being the Annual Meeting of the Association.

Informal luncheons are held at 12.15 P. M. every Wednesday, and the place of meeting may be ascertained from the Secretary.

The by-laws of the Association provide for the extension of hospitality to any member of the Society who may be temporarily in Los Angeles, and any such member will be gladly welcomed as a guest at any of the meetings or luncheons.

Spokane Association, Organized 1914.

J. C. Ralston, President; B. J. Garnett, Secretary, City Hall, Spokane, Wash.

The regular meetings of the Association are held on the second Friday of each month, except July and August. The hour and place of meeting are not fixed, but this information will be furnished on application to the Secretary.

Visiting members are invited to attend the meetings and luncheons.

(Abstract of Minutes of Meeting)

June 8th, 1917.—The meeting was called to order at the University Club; President Ralston in the chair; B. J. Garnett, Secretary.

The report of the Committee on the Post Street Bridge Failure was presented and adopted.

The question of the relations between Local Associations and the American Society of Civil Engineers was discussed.

Adjourned.

Texas Association, Organized 1913.

John B. Hawley, President; J. F. Witt, Secretary, Dallas, Tex.

Utah Association, Organized 1916.

George L. Swendsen, President; H. S. Kleinschmidt, Secretary-Treasurer, 306 Dooly Building, Salt Lake City, Utah.

The Annual Meeting of the Association is held on the first Wednesday in April. The time of other meetings is not fixed, but this information will be furnished on application to the Secretary.

(Abstract of Minutes of Meeting)

April 4th, 1917.—The Annual Meeting was called to order at the Newhouse Hotel, Salt Lake City, Utah; Vice-President A. F. Doremus in the chair; H. S. Kleinschmidt, Secretary; and present, also, 75 members and guests.

Ballots for officers for 1917 were canvassed, and resulted in the election of George L. Swendson, President, and R. C. Gemmell, Second Vice-President. A complete list of the officers of the Association for 1917 is as follows: George L. Swendsen, President; A. F. Doremus, First Vice-President; R. C. Gemmell, Second Vice-President; H. S. Kleinschmidt, Secretary-Treasurer.

Various matters of interest were discussed, and in the evening a joint meeting was held with the Utah Society of Engineers. After an

informal dinner, a stereopticon lecture was given by J. B. Lippincott, M. Am. Soc. C. E., on the "Los Angeles Aqueduct", and C. F. Brown, Assoc. M. Am. Soc. C. E., addressed the meeting on "Some Unusual Problems in Drainage."

Adjourned.

MINUTES OF MEETINGS OF SPECIAL COMMITTEES TO REPORT UPON ENGINEERING SUBJECTS

Special Committee to Codify Present Practice on the Bearing Value of Soils for Foundations, etc.

January 21st, 1917.—The meeting was called to order at the office of Allen Hazen, M. Am. Soc. C. E., New York City. Present Robert G. Cummings (Chairman), Edwin Duryea, E. G. Haines, Allen Hazen, and Walter J. Douglas (Secretary).

The past work of the Committee was discussed, and general plans for the future were made.

Mr. Hazen suggested that, after the Classification of Soils had been determined, and during the completion of such Classification, the Committee consider its work on the basis of character of soil troubles. After discussing the subject it was agreed that failures would be considered along the following lines, which would be used as a general guide:

(1).—Compression of the soils, divided generally as follows:
- (*a*) Breaking edges of the grain,
- (*b*) Squeezing out of water,
- (*c*) Result of organic inclusions.

(2).—Second general classification of failure:
Slipping. Quicksand, lubricants, etc.

(3).—Third general classification of failure:
Chemical changes.

It was also agreed that Mr. Hazen submit a proposed classification along these lines at an early date.

The work of the Committee for the coming year is to include a development of the subject along the lines of failure, as indicated in the foregoing classification, as well as definitions of soils, methods of analyses, and reports from the Sub-Committee on Earth Pressures, including apparatus for testing the bearing capacity of soils, a report of the Sub-Committee on Piles, and a report on existing data on the bearing capacity of earths.

The Committee also considered whether the word "earth" should be substituted for the word "soils", that is, "the bearing capacity of earth" instead of "the bearing capacity of soils."

April 19th, 1917.—The meeting was called to order at the House of the Society at 10.45 A. M. Present, Robert G. Cummings (Chairman), Edwin Duryea, and E. G. Haines (Secretary *pro tem.*).

The minutes of the meeting of January 21st, 1917, were read, and after some discussion of the last paragraph, were approved as

amended. It was agreed to retain the word "soils" as defined in the Committee's Report presented to the Annual Meeting of January 19th, 1916.

Mr. Haines reported that since November, 1916, Sub-Committee "E" had sent out 1 200 letters to engineers, requesting records of tests for lateral pressures, with the following results:

Letters sent out		1 200
No replies	1 004	
Replies with no records	131	
Replies suggesting possible sources of information	52	
Replies with information furnished	13	
Totals	1 200	1 200

After discussion, it was decided not to continue the investigation any further, at least at present, except to follow up the replies suggesting possible sources of information.

The Committee, it was reported, had on hand considerable information received at various times from various sources, much of which is of great value. It was decided that abstracts of such information should be prepared by the Secretary to be printed as an Appendix to the Progress Report of 1917.

Mr. Haines presented the results of some experiments with the soil testing apparatus furnished by Mr. Cummings, and stated that, judging from the results of these tests, and after certain changes had been made which would permit of a greater range of pressures and bearing areas, and also provide for applying pressures over a longer interval of time, the instrument was capable of being developed into a practical form which could be used to advantage.

Mr. Cummings presented a letter dated April 6th, 1917, from John H. Griffith, M. Am. Soc. C. E., Chairman of the Sub-Committee on Soils of the Bureau of Standards. This letter contained much that was of interest relative to the work of the Committee, and the Chairman was instructed to send copies of it to each member of the Board of Direction of the Society.

Mr. Cummings reported that Professor J. Hammond Smith, of the University of Pittsburgh, would design, at no cost to the Society, a full-sized testing apparatus, similar in principle to his smaller apparatus, for determining pressures on granular materials.

The matter of practical field tests for determining the bearing value of soils was discussed, and it was decided that such tests should be made on the basis of 1 sq. ft. of area, the testing area to be in the form of a circle, and that the tested areas should be below the surrounding surface, or otherwise restrained. Mr. Haines agreed to prepare a tentative sketch of a practical field testing apparatus to comply with the requirements.

Mr. Hazen's paper on the reasons for soil failures was discussed in detail and certain changes were suggested. It was then agreed that copies of all written discussions by members of the Committee

should be sent direct to each member of the Committee, including the President of the Society.

The meeting was adjourned at 6 P. M. subject to the call of the Chair.

Special Committee on Materials for Road Construction

May 5th, 1917.—The meeting was called to order at 9.15 A. M.; at the House of the Society. Present, W. W. Crosby (Chairman), A. W. Dean, Charles J. Tilden, George W. Tillson, and A. H. Blanchard (Secretary).

The minutes of the meeting of February 22d, 1917, were read and approved.

Plans for the drafting of a Final Report to be presented at the Annual Meeting of the Society in January, 1918, were adopted, and fourteen sub-committees were appointed to prepare preliminary drafts of the several sub-divisions of the report.

On motion, the Committee adjourned to meet at 9 A. M., on Saturday, May 19th, 1917, at the House of the Society.

May 19th, 1917.—The meeting was called to order at the Society House at 9.30 A. M. Present, W. W. Crosby (Chairman), H. K. Bishop, A. W. Dean, Nelson P. Lewis, and A. H. Blanchard (Secretary).

The minutes of the meeting of May 5th, 1917, were approved.

On motion, duly seconded, it was decided that the Reports of Sub-Committees on Introduction, General Principles Concerning Materials and Their Use to be Observed in Framing Specifications for Pavements and Road Crusts, Asphalt Block Pavements, Bituminous Concrete Pavements, and Bituminous Macadam Pavements, be accepted tentatively, be referred back to the several sub-committees, be revised, including the addition of amendments adopted, and that copies of the revised reports be forwarded by each Sub-Committee to each member of the Committee not later than one week prior to the date to be hereafter set for the consideration of the final draft of the 1918 Report.

On motion, duly seconded, the Committee adjourned to meet at 9 A. M., on Monday, June 18th, 1917, at the House of the Society.

June 18th, 1917.—The meeting was called to order at the House of the Society at 9.45 A. M. Present, W. W. Crosby (Chairman), A. W. Dean, Nelson P. Lewis, and A. H. Blanchard (Secretary).

The minutes of the meeting of May 19th, 1917, were approved.

On motion, duly seconded, it was ordered that the following reports of Sub-Committees be tentatively accepted, be referred back to the several sub-committees, be revised, including the addition of amendments adopted, and that copies of the revised reports be forwarded by each Sub-Committee to each member of the Committee not later than one week prior to the date to be hereafter set for the consideration of the final draft of the 1918 Report:

Report on "Brick Pavements"; report on section of "General Conclusions" covering "Bituminous and Non-Bituminous Materials"; report on "Broken Stone Roads"; report on "Earth and Sand-Clay Roads"; and report on "'Sheet-Asphalt Pavements".

On motion, duly seconded, it was ordered that each report on a specific type of road or pavement begin with an introductory paragraph containing references to those sections under "General Conclusions" which pertain to details of construction of the given type of road or pavement to which the report refers.

On motion, duly seconded, it was ordered that the Sub-Committee on Broken Stone Roads with Bituminous Surfaces be discontinued, and that Mr. A. W. Dean be appointed a sub-committee on Bituminous Surface Treatments, and be requested to submit the report at the next meeting of the Committee.

On motion, the Committee adjourned to meet at 9 A. M. on Monday, July 9th, 1917, at the House of the Society.

July 9th, 1917.—The meeting was called to order at the House of the Society, at 9.45 A. M. Present, Nelson P. Lewis (Chairman *pro tem.*), Charles J. Tilden, George W. Tillson, and Arthur H. Blanchard (Secretary).

The minutes of the meeting of June 18th, 1917, were approved.

On motion, duly seconded, it was decided that the Reports of the Sub-Committees on Cement-Concrete Pavements, Gravel Roads, Stone Block Pavements, Wood Block Pavements, Analyses and Tests of Bituminous Materials, and Analyses and Tests of Non-Bituminous Materials, be tentatively accepted, referred back to the several sub-committees, be revised, including the addition of amendments adopted, and that copies of the revised reports be forwarded by each Sub-Committee to each member of the Committee not later than one week prior to the date to be hereafter set for the consideration of the final draft of each Sub-Committee report.

On motion, duly seconded, it was agreed that the Sub-Committee on Forms be requested to present, in its Final Report, one form, to be an historical record of construction applicable to all types of roads and pavements and similar in character to that included as Appendix A in the Progress Report of the Committee for 1916.

On motion, duly seconded, it was decided that the order of business for the next meeting of the Committee be as follows: Preliminary report of the Sub-Committee on Bituminous Surface Treatments; Preliminary reports of the Sub-Committees on Definitions; Final report of the Sub-Committee on Forms; Final report of the Sub-Committee on Introduction, General Conclusions and General Principles.

On motion, duly seconded, the Committee adjourned to meet at 9.30 A. M., on Monday, August 27th, 1917, at the House of the Society.

Special Committee on Steel Columns and Struts

June 18th, 1917.—The meeting was called to order at the House of the Society at 10.15 A. M. Present, Lewis D. Rights (Chairman), Joseph R. Worcester, George F. Swain, and Clarence W. Hudson (Secretary). Professor Nelson, of the Bureau of Standards, was also present.

Professor Nelson reported that 28 specimen tests of the material of the tested columns had been made since the meeting of January

17th. This number, with those made previous to that meeting, gives a total of 130 tests of such column material.

Professor Nelson also stated that the Bureau was very busy with Government work, and that little, if any, work could be expected on the Committee's supplementary tests.

The Committee requested Professor Nelson to look up the reports, of the Bureau of Standards in Pittsburgh, on the material used in columns, Types 1, 1*a*, 4, 4*a*, 5, 5*a*, and 5*b*, and see if it met the specifications.

On motion, duly seconded, the work of the Sub-Committee, consisting of the Chairman and Secretary, in preparing the drawing of the stress-strain curves, showing the determination of the useful limit point of the columns tested, was approved.

On motion, duly seconded, the Chairman and Secretary were instructed to draw up a final report and submit it to the members of the Committee for discussion and approval.

PRIVILEGES OF ENGINEERING SOCIETIES EXTENDED TO MEMBERS OF THE AMERICAN SOCIETY OF CIVIL ENGINEERS

Members of the American Society of Civil Engineers will be welcomed by the following Engineering Societies, both to the use of their Reading Rooms, and at all meetings:

American Institute of Electrical Engineers, 25 West Thirty-ninth Street, New York City.

American Institute of Mining Engineers, 25 West Thirty-ninth Street, New York City.

American Society of Mechanical Engineers, 25 West Thirty-ninth Street, New York City.

Assoçiacão dos Engenheiros Civis Portuguezes, Lisbon, Portugal.

Australasian Institute of Mining Engineers, Melbourne, Victoria, Australia.

Boston Society of Civil Engineers, 715 Tremont Temple, Boston, Mass.

Brooklyn Engineers' Club, 117 Remsen Street, Brooklyn, N. Y.

Canadian Society of Civil Engineers, 176 Mansfield Street, Montreal, Que., Canada.

Civil Engineers' Society of St. Paul, St. Paul, Minn.

Cleveland Engineering Society, Chamber of Commerce Building, Cleveland, Ohio.

Cleveland Institute of Engineers, Middlesbrough, England.

Dansk Ingeniorforening, Amaliegade 38, Copenhagen, Denmark.

Detroit Engineering Society, 46 Grand River Avenue, West, Detroit, Mich.

Engineering Association of Nashville. Commercial Club Building, Nashville, Tenn.

Engineers and Architects Club of Louisville, 1412 Starks Building, Louisville, Ky.

Engineers' Club of Baltimore, 6 West Eager Street, Baltimore, Md.

Engineers' Club of Kansas City, E. B. Murray, Secretary, 920 Walnut Street, Kansas City, Mo.

Engineers' Club of Minneapolis, 17 South Sixth Street, Minneapolis, Minn.

Engineers' Club of Philadelphia, 1317 Spruce Street, Philadelphia, Pa.

Engineers' Club of St. Louis, 3817 Olive Street, St. Louis, Mo.

Engineers' Club of Toronto, 96 King Street, West, Toronto, Ont., Canada.

Engineers' Club of Trenton, Trent Theatre Building, 12 North Warren Street, Trenton, N. J.

Engineers' Society of Northeastern Pennsylvania, 415 Washington Avenue, Scranton, Pa.

Engineers' Society of Pennsylvania, 31 South Front Street, Harrisburg, Pa.

Engineers' Society of Western Pennsylvania, 568 Union Arcade Building, Pittsburgh, Pa.

Florida Engineering Society, J. R. Benton, Secretary, Gainesville, Fla.

Institute of Marine Engineers, The Minories, Tower Hill, London, E., England.

Institution of Civil Engineers, Great George Street, Westminster, S. W., London, England.

Institution of Engineers of the River Plate, Calle 25 de Mayo 195, Buenos Aires, Argentine Republic.

Institution of Naval Architects, 5 Adelphi Terrace, London, W. C., England.

Junior Institution of Engineers, 39 Victoria Street, Westminster, S. W., London, England.

Koninklijk Instituut van Ingenieurs, The Hague, The Netherlands.

Louisiana Engineering Society, State Museum Building, Chartres and St. Ann Streets, New Orleans, La.

Memphis Engineers' Club, Memphis, Tenn.

Midland Institute of Mining, Civil and Mechanical Engineers, Sheffield, England.

Montana Society of Engineers, Butte, Mont.

North of England Institute of Mining and Mechanical Engineers, Newcastle-upon-Tyne, England.

Oregon Society of Civil Engineers, Portland, Ore.

Pacific Northwest Society of Engineers, 803 Central Building, Seattle, Wash.

Rochester Engineering Society, Rochester, N. Y.

Sociedad Colombiana de Ingenieros, Bogota, Colombia.

Sociedad de Ingenieros del Peru, Lima, Peru.

Societe des Ingenieurs Civils de France, 19 rue Blanche, Paris, France.

Society of Engineers, 17 Victoria Street, Westminster, S. W., London, England.

Svenska Teknologforeningen, Brunkebergstorg 18, Stockholm, Sweden.

Tekniske Forening, Vestre Boulevard 18–1, Copenhagen, Denmark.

Vermont Society of Engineers, George A. Reed, Secretary, Montpelier, Vt.

Western Society of Engineers, 1737 Monadnock Block, Chicago, Ill.

ACCESSIONS TO THE UNITED ENGINEERING SOCIETY LIBRARY

(From April 3d to July 10th, 1917)

DONATIONS*

The statements made in these notices are taken directly from the book itself, and this Society is not responsible for them.

DOCUMENTS GOVERNING THE CONSTRUCTION OF A BRIDGE:

Including a Reprint of the Specifications, Proposal, Contract and Bond of the Columbia River Interstate Bridge, a Description of the Structure, and a Discussion of the Function of Specifications. By E. E. Howard. N. Y., John Wiley & Sons; Lond., Chapman & Hall, Ltd., 1916. 113 pp., 8 x 11 in., 1 pl., paper. $1.00.

These specifications have been printed in book form with the thought that they may be of value in suggesting satisfactory substance and arrangement for like documents, being the product, the author states, of a variety of experiences, of technical opinions and engineering judgments reached only after years of practice.

SPONS' ELECTRICAL POCKET-BOOK:

A Reference Book of General Electrical Information, Formulae and Tables for Practical Engineers. By Walter H. Molesworth. N. Y., Spon & Chamberlain; Lond., E. & F. N. Spon, Ltd., 1916. 488 pp., 4 x 7 in., 325 illus., cloth. $2.00.

This book has been written for practical engineers, and it has been the aim to treat all subjects concisely and to avoid intricate mathematics. Full metric conversion tables have been inserted.

ELECTRICAL MEASURSEMENTS IN PRACTICE.

By Malcolm Farmer. N. Y., McGraw-Hill Book Co., Inc.; Lond., Hill Publishing Co., Ltd., 1917. 12 + 359 pp., 230 illus., 6 x 9 in. $4.00.

Intended to present the subject in a simple practical manner, from the point of view of engineers actively engaged in making measurements, tests and investigations in the electrical industry. All classes of measurements ordinarily made in the laboratory or by the testing engineer are included.

CENTRAL-STATION ELECTRIC SERVICE:

Its Commercial Development and Economic Significance as Set Forth in the Public Addresses (1897-1914) of Samuel Insull. Edited, with an Introduction, by William Eugene Kelly. Chic., privately printed, 1915. 39 + 495 pp., 9 x 6 in., 105 illus., 36 pl., cloth. (Gift of the author.)

A collection of forty speeches bearing on central station electric service, delivered by the author between 1897 and 1914. The editor states that these addresses are the work of one who has led the way to new conceptions of the economic functions of central station electric service, and that much information of historical value, some of it never before published, is scattered through the book.

THE POWER KINK BOOK:

Novel Ideas and Simple Devices for Meeting Emergencies in the Power Plant, Compiled from the Regular Issues of *Power*. N. Y., Power, 1917. 146 pp., 147 illus., 6 x 9 in., boards. $1.75.

A collection of methods devised by power plant engine runners and machinists for making repairs, overcoming difficulties, and preventing accidents. The methods are classified into appropriate groups and clearly illustrated by numerous drawings and sketches.

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* Unless otherwise specified, books in this list have been donated by the publishers.

ARTIFICIAL ELECTRIC LINES:

Their Theory, Mode of Construction and Uses. By A. E. Kennelly. N. Y., McGraw-Hill Book Co., Inc.; Lond., Hill Publishing Co., Ltd., 1917. 348 pp., 117 illus., 6 x 9 in., cloth. $4.00. (Gift of the author.)

Intended as a text-book for engineering-laboratory students and as a reference book for students of electric transmission in general. Based on the author's "The Application of Hyperbolic Functions to Electrical Engineering Problems." Restricted almost entirely to the phenomena of the steady state.

STORAGE BATTERIES SIMPLIFIED:

Operating Principles—Care and Industrial Applications; a Complete, Non-technical but Authoritative Treatise Discussing the Development of the Modern Storage Battery, Outlining the Basic Operation of the Leading Types, also the Methods of Construction, Charging, Maintenance and Repair. All Practical Applications of Commercial Batteries are Shown and Described. By Victor W. Page. 208 pp., 89 illus., 5 x 8 in., cloth. $1.50.

Special instructions for care and repair of automobile batteries and a glossary of terms are included.

TELEGRAPHY:

A Detailed Exposition of the Telegraph System of the British Post Office. 3d ed. rev. and enl. N. Y., The Macmillan Co.; Lond., Whittaker & Co., 1916. 20 + 985 pp., 630 illus., 7½ x 5 in., cloth. $3.50.

Intended as a detailed exposition of the telegraph practice of the British Post Office and for students. Mathematics is avoided. This edition has been revised to include the latest practice and to correct errors.

A WONDERFUL FIFTY YEARS.

By Edwin T. Holmes. Privately published (copyright 1917). 133 pp., 55 illus., 6 x 9 in., cloth. (Gift of the author.)

Personal reminiscences of the development of electrical communication from 1866 to 1916, during which time the author has been connected with the telegraph, telephone and electrical protective signaling industries of the country. Contains many interesting illustrations, reproductions of documents, and letters of historic interest.

THE NAVAL ARCHITECTS' AND SHIPBUILDERS' POCKET-BOOK

Of Formulae, Rules, and Tables: And Marine Engineers' and Surveyors' Handy Book of Reference. By Clement Mackrow and Lloyd Woodland. 11th ed. thoroughly rev., with a Section on Aeronautics. N. Y., The Norman W. Henly Publishing Co., 1916. 12 + 742 pp., 6 x 4 in., 150 illus., flexible leather. $5.00.

The continual development of the science of naval architecture and the tendency towards standardization and regulation of parts of the structure and equipment of ships have created a need, the authors state, for a new edition of this work. Their object has been, as in former editions, to condense into a compact form all the data and formulas ordinarily required by the shipbuilder or naval architect. A new section on Speed and Horse-Power has been inserted, together with a brief description of modern methods of powering and determining forms suitable from a propulsive standpoint; the sections on Strength of Materials, Riveted Joints and Stresses in Ships have been considerably extended; information on British standard sections, screws, keys, etc., has been added as well as two new sections on aeronautical matters. The remaining subjects treated, which were also in previous editions, have been brought completely up to date.

PRACTICAL MARINE ENGINEERING;

For Marine Engineers and Students, with Aids for Applicants for Marine Engineers' Licenses. By Capt. C. W. Dyson. 4th ed. rev.

and enl. N. Y., *Marine Engineering,* 1917. 16 + 982 pp., 500 illus., 6 x 9 in., cloth. $6.00.

A simple and fairly complete treatise, intended for operative engineers, and hence paying especial attention to the construction, operation, management, and care of marine machinery. The use of higher mathematics is avoided.

MOTOR BOATS, HYDROPLANES, HYDROAEROPLANES;

Construction and Operation, with Practical Notes on Propeller Calculation and Design; An Illustrated Manual of Self Instruction for Owners and Operators of Marine Gasoline Engines and Amateur Boat-Builders. By Thomas H. Russell, with Revisions and Extensions by John B. Rathbun. Chic., Charles C. Thompson Co., 1917. 254 pp., 106 illus., 5 x 8 in., cloth. $1.00.

Practical, non-mathematical handbook. Directions for building motor-boats are given, together with information concerning the various available types of motors and detailed instruction for their installation and operation.

MECHANICAL MOVEMENTS, POWERS, AND DEVICES;

A Treatise Describing Mechanical Movements and Devices Used in Constructive and Operative Machinery and the Mechanical Arts, Being Practically a Mechanical Dictionary, Commencing with a Rudimentary Description of the Early Known Mechanical Powers and Detailing the Various Motions, Appliances and Inventions Used in the Mechanical Arts to the Present Time. Including a Chapter on Straight Line Movements. By Gardner D. Hiscox. N. Y., The Norman W. Henley Publishing Co., 1917. 409 pp., 1 800 illus., 6 x 9 in., cloth. $3.00.

This edition is enlarged by the addition of over one hundred and sixty new mechanical movements and devices, and contains a total of sixteen hundred and sixty-five examples.

MECHANICAL APPLIANCES, MECHANICAL MOVEMENTS AND NOVELTIES OF CONSTRUCTION.

By Gardner D. Hiscox. 4th ed. enl. N. Y., The Norman W. Henley Publishing Co., 1917. 396 pp., 1 000 illus., 6 x 9 in., cloth. $3.00.

A companion volume to the author's "Mechanical Movements, Powers and Devices," in which the special requirements of various arts and manufactures are considered, and more detailed explanations of the devices are given. Includes a chapter on perpetual motion, illustrating many types of perpetual motion machines.

STEAM POWER.

By C. F. Hirshfeld and T. C. Ulbricht. N. Y., John Wiley & Sons, Inc.; Lond., Chapman & Hall, Ltd., 1916. 8 + 420 pp., 8 x 5 in., 91 illus., cloth. $2.00.

An attempt to collect in a comparatively small book such parts of the field of steam power as should be familiar to engineers whose work does not require that they be conversant with the more complicated thermodynamic principles considered in advanced treatments. Mathematical treatment of the subject has been eliminated to the greatest possible extent. Intended for use as a text-book by students of civil engineering and in teaching power plant operators.

STEAM PIPING:

Its Economical Design and Correct Layout. By A. Langstaff Johnston, Jr. N. Y., The Engineering Magazine Co., 1916. 6 + 62 pp., 5 x 8 in., 3 illus., 4 diagrams, cloth. $2.00.

Consolidated and revised from a series of articles published in the *Engineering Magazine* in 1915. Analyzes the factors governing the flow of steam in pipes, and presents a group of curves for use in solving the problems of practical installation

and determining the most economical size of pipe to select for any given conditions. Contents: How to Find the Right Pipe Sizes; Special Conditions Affecting Low-Pressure Systems; and Savings Obtainable from Exhaust Steam.

STEAM TURBINES:

A Practical Work on the Development, Advantages and Disadvantages of the Steam Turbine; The Design, Selection, Operation, and Maintenance of Steam Turbine and Turbo-Generator Plants. By Walter S. Leland. Chic., American Technical Society, 1917. 137 pp., 116 illus., 6 x 8 in., cloth. $1.00.

Intended for those less interested in the finer points of steam turbine theory than in the results accomplished and the way in which they have been secured by the successful builders. Over one-half of the book is devoted to descriptions of various commercial turbines.

STEAM TURBINES:

A Treatise Covering U. S. Naval Practice. By G. J. Myers. Annapolis, The U. S. Naval Institute, 1917. 7 + 246 pp., 12 x 8 in., 179 illus., 23 diagrams, 9 pl., cloth. $4.50.

This book has been prepared to meet the needs of an elementary treatise on steam turbines for the use of midshipmen at the U. S. Naval Academy, and deals mainly with types found in the U. S. Naval Service.

HIGH SPEED INTERNAL COMBUSTION ENGINES.

By Arthur W. Judge. N. Y., The Macmillan Co.; Lond., Whittaker & Co., 1916. 9 + 350 pp., 217 illus., 6 x 9 in., cloth. $5.50.

The author has collected and classified the available information, and presents it as briefly as possible. The work discusses the theory of high speed internal combustion engines, and the experimental results which have been obtained. Contents: The Thermodynamics of the Internal Combustion Engine; The Conditions Occurring in Actual Engines; Pressures and Temperatures in Internal Combustion Engines; Indicators and Indicator Diagrams; The Mechanics of the High-Speed Internal Combustion Engine; Engine Balance.

GAS CHEMISTS' HANDBOOK.

Compiled by Technical Committee, Sub-Committee on Chemical Tests, 1916, of the American Gas Institute. C. C. Tutweiler, Chairman; A. F. Kunberger, Editor. N. Y., American Gas Institute. 354 pp., 9 x 6 in., 70 illus., cloth. $3.50.

The present handbook, a revision of the one compiled in 1914, presents methods for sampling and testing the materials used in gas manufacture. Contents: Raw Materials; Products of Gas Manufacture; Impurities in Gas; Tar Products; Miscellaneous and Tables.

HANDBOOK OF CASINGHEAD GAS.

By Henry P. Westcott. Erie, Metric Metal Works, 1916. 9 + 274 pp., 5 x 8 in., illus., cloth. $2.50.

Methods, statistics, etc., intended to supply information on the processes used for extracting gasoline from natural gas. Based on visits to many existing plants and a study of their reports. Contents: General Physical Properties of Casinghead Gas Wells; Construction of Pipe Lines; Measuring Casinghead Gas; Gasoline Plant, Compression Method; Gasoline Plant, Absorption Method; Transportation of Gasoline; Miscellaneous.

GASOLINE AND HOW TO USE IT.

By G. A. Burrell. Bost., Oil Statistical Society, Inc., 1916. 281 pp., 6 x 4½ in., 1 illus., cloth. $1.50.

The preface states that the aim of this book is to provide an intelligent understanding of the use of gasoline, to assist the motorist and the farmer, and to give the history of gasoline and petroleum.

MODERN MILLING:

A Practical Manual on Milling Machines, Milling Accessories and Milling Operations. By Ernest Pull. N. Y., The Macmillan Co.; Lond., Whittaker & Co., 1917. 8 + 207 pp., 188 illus., 6 x 9 in., cloth. $3.00.

Universal and plain milling machines are described, as well as several special types, and their use for indexing, gear cutting and other operations is explained. Includes a chapter on speeds and feeds.

PLAIN AND ORNAMENTAL FORGING.

By Ernst Schwarzkopf. N. Y., John Wiley & Sons, Inc.; Lond., Chapman & Hall, Ltd., 1916. 10 + 267 pp., 228 illus., 5 x 8 in. $1.50.

Written to provide a really practical treatise on the theory and practice of art metal and blacksmith work, suitable for use as a text-book for beginners. Simple and detailed drawings illustrating each important operation are provided, together with full explanations. Intended especially for self-instruction. The author has had many years experience as a blacksmith and an instructor in forge work.

QUESTIONS AND ANSWERS RELATING TO MODERN AUTOMOBILE DESIGN, CONSTRUCTION, DRIVING AND REPAIR:

A Practical Treatise Consisting of Thirty-nine Lessons in the Form of Questions and Answers Written with Special Reference to the Requirements of the Non-Technical Reader Desiring Easily Understood Explanatory Matter Relating to All Branches of Automobiling. By Victor W. Page. Rev. and enl. ed. N. Y., The Norman W. Henley Publishing Co., 1917. 15 + 701 pp., 5 x 8 in., 397 illus., 3 pl., cloth. $1.50.

FARM MOTORS:

Steam and Gas Engines, Hydraulic and Electric Motors, Traction Engines, Automobiles, Animal Motors, Windmills. By Andrey Potter. 2d ed. rev. and enl. (Agricultural Engineering Series). N. Y., McGraw-Hill Book Co., Inc.; Lond., Hill Publishing Co., Ltd., 1917. 11 + 299 pp., 296 illus., 6 x 8 in., cloth. $1.50.

Includes the fundamental principles governing their construction, management and working. Intended for students of agricultural engineering and farmers. In the present edition the chapters on gas and oil engines and on traction engines have been enlarged, those on the steam engine rewritten, and chapters added on automobiles and animal motors.

THE MECHANISM OF THE LINOTYPE:

A Complete and Practical Treatise on the Installation, Operation and Care of the Linotype, for the Novice as well as the Experienced Operator. By John S. Thompson. 6th ed. rev. Chic., The Inland Printer Co., 1916. 10 + 280 pp., 7 x 5 in., 77 illus., 1 por., leather. $2.00.

The present edition of this text-book embodies all the improvements made in the linotype to the present time. A brief sketch is given of Ottmar Mergenthaler, the inventor of the linotype.

A HANDBOOK OF BRIQUETTING.

By G. Franke. Translated by Fred C. A. H. Lantsberry. Vol. 1, The Briquetting of Coals, Brown Coals, and Other Fuel. Phila., J. B. Lippincott Co.; Lond., Charles Griffin and Co., Ltd., 1917. 19 + 631 pp., 9 x 6 in., 225 illus., 9 pl., cloth. $9.00.

This book is not only intended as a handbook for those engaged in the industry, but can be used as a text-book in schools of mining and metallurgy and technical

high schools. The translator adds that the object of the translation is to put in the possession of English industrial leaders, knowledge which other countries have been compelled to acquire.

COMPRESSED AIR FOR THE METAL WORKER.

By Charles A. Hirschberg. N. Y., The Clark Book Co., 1917. 321 pp., 294 illus., 5 x 8 in., cloth. $3.00.

Describes the various purposes for which compressed air is used in power plants, foundries, machine shops, forge shops, etc. The various types of tools and machines are illustrated, as well as the commercial forms of air compressors. A compendium of present-day methods of utilizing compressed air, confined entirely to practice and omitting theory.

THE WORLD'S MINERALS.

By Leonard J. Spencer, with an Appendix by W. D. H. Hamman. N. Y., Frederick A. Stokes Co., 1916. 11 + 327 pp., 8 x 6 in., 40 pl., 21 diagrams, cloth. $2.75.

An attempt to present a popular and readable account, in the main confined to the 116 species of the more common simple minerals, which will help the student collector to identify his own specimens. Forty color-plates are included.

CHEMICAL TESTS FOR MINERALS.

By Arthur J. Burdick. Beaumont, Cal., The Gateway Publishing Co., 1917. 93 pp., 5 x 8 in., cloth. $1.25.

Handbook of simple qualitative tests, intended to enable prospectors without chemical training to identify the various rocks and ores met with in the field.

MICROSCOPIC EXAMINATION OF STEEL.

By Henry Fay. (Wiley Engineering Series No. 3.) N. Y., John Wiley & Sons; Lond., Chapman & Hall, Ltd., 1917. 18 pp., 9 x 6 in., 32 pl., cloth. $1.25.

The material contained in this volume was originally issued by the Ordnance Department, U. S. A., and was intended for the exclusive use of inspectors of ordnance material, but is now published for the use of others interested in the inspection of steel. It is meant only to present a mere outline of metallographic methods illustrating typical examples, but is not for use as a text-book. It is intended particularly for those who are in need of help in the interpretation of results. Over two-thirds of the volume consists of full-page plates.

THE STORY OF BETHLEHEM STEEL.

By Arundel Cotter. N. Y., The Moody Magazine and Book Co., 1916. 65 pp., 7½ x 5 in., 8 pl., cloth. 75 cents.

Contents: Town Founded by Moravian Colonists; When Schwab Went Down to Bethlehem; Bethlehem and the War Stock Boom; Schwab's Theories.

TUBE MILLING:

A Treatise on the Practical Application of the Tube Mill to Metallurgical Problems. By Algernon Del Mar. N. Y., McGraw-Hill Book Co., Inc.; Lond., Hill Publishing Co., Ltd., 1917. 10 + 159 pp., 6 x 9 in., 70 illus., 1 pl., cloth. $2.00.

Covers the use of the conical and cylindrical tube mills for grinding ores, indicating in detail the best means of obtaining capacity at the least cost, and describing recent installations. It is, the author states, the only book entirely devoted to the subject. Contents: General Description; Amalgamating with the Tube; Grinding Ores with the Tube Mill for Flotation; Crushing Efficiencies; The Use of Wrought Iron and Alloy Steel; Appendix.

FRYE'S TABLES FOR ASCERTAINING THE VALUE OF GOLD-QUARTZ SPECIMENS.

By Jason S. Frye. Downieville, Cal., Jason S. Frye (privately printed), 1916. 56 pp., 4 x 3 in., leather. $1. (Gift of the author.)

Vest-pocket book of tables for finding the percentage of gold and the value per ounce from the specific gravity of gold-quartz specimens containing different amounts of gold.

STRESSES IN WIRE-WRAPPED GUNS AND IN GUN CARRIAGES.

By Lieut. Col. Colden L'H. Ruggles. N. Y., John Wiley & Sons, Inc.; Lond., Chapman & Hall, Ltd., 1916. 11 + 259 pp., 9 x 6 in., cloth. $3.00.

Explains a number of the important engineering principles underlying the design of wire-wrapped guns and of gun carriages. Originally prepared for the use of the cadets of the U. S. Military Academy. Contents: Elastic Strength of Wire-Wrapped Guns; Determination of the Forces Brought upon the Principal Parts of the 3-Inch Field Carriage and a Disappearing Gun Carriage by the Discharge of the Gun; Stresses in Parts of Gun Carriages; Toothed Gearing; Counter Recoil Springs.

BATTLE FIRE TRAINING.

By Capt. G. S. Turner and Capt. J. J. Fulmer. Menasha, Wis., George Banta Publishing Co. 294 pp., 47 illus., 4 x 7 in., cloth. $1.00.

Calls attention to the existing necessity for the adoption in our army of a uniform system of collective training in battle fire, and offers a system based upon the principles laid down in the manuals upon fire and fire tactics published by the War Department.

ENGINEERING ANALYSIS OF A MINING SHARE.

By J. C. Pickering. N. Y., McGraw-Hill Book Co., Inc.; Lond., Hill Publishing Co., Ltd., 1917. 8 + 95 pp., 9 x 6 in., cloth. $1.50. (Gift of the author.)

An endeavor to set forth the considerations which enter logically into the analysis of a mining venture, approached from an engineering viewpoint. Contents: Classification of Mining Properties; Development Companies; Elements to be Considered in the Analysis of a Mining Report; Operating Profits *vs.* Dividends; Determination of the Value of a Mine or Mining Share; Valuing Mine Products; World's Gold Production with a Review of the World's Greatest Gold-field; Mining *vs.* Industrial and other Types of Investments with an Analysis of the Affairs of Typical Mining Companies.

COMPRESSED AIR PRACTICE IN MINING.

By David Penman. Phila., J. B. Lippincott Co.; Lond., Charles Griffin & Co., Ltd., 1917. 221 pp., 113 illus., 5 x 8 in., cloth.

Text-book for use in mining schools, but also useful, it is hoped, to those in practice. Discusses air-compressors, methods of transmitting power and the use for compressed air for coal cutting, conveyors, drills, etc. A comparison of the advantages of compressed air and other means of transmitting power in mines is given.

MINE GASES AND VENTILATION:

A Reference Handbook Combining Theory and Practice of Coal Mining: Designed to Meet the Needs of all Students of Coal Mining, Including Mining Engineers, Mine Superintendents, Foremen, Firebosses, Shotfirers, and Miners Preparing for Examination for Certificates of Competency. By J. T. Beard. N. Y., Hill Publishing Co., 1916. 206 pp., 7 x 5 in., flexible cloth. $2.00. (Gift of the author.)

Composed of material on the atmosphere, gases and ventilation of mines, which has appeared in pocket-book form, in the "Study Course in Coal Mining" Department of *Coal Age* since March, 1913, and which was prepared in response to requests from coal miners who wished to know the development of formulas, the explanation of principles and the most approved and generally adopted methods. Contents: Air; Heat; Mine Gases; Theory of Ventilation; Practical Ventilation; Addenda.

PRINTING TRADES BLUE BOOK 1917.

N. Y. and Chic., A. F. Lewis & Co., 1917. 543 pp., 6 x 8 in., cloth. $3.00.

Directory of printers, dealers in paper machinery, metals, type, etc. in New York and surrounding towns. Arranged alphabetically, by telephone numbers and by classes. Includes directories of paper brands and of printing associations, unions and clubs.

HENDRICKS' COMMERCIAL REGISTER OF THE UNITED STATES:

For Buyers and Sellers. Twenty-fifth Annual Edition. N. Y., S. E. Hendricks Co., Inc., 1916. 1738 pp., 8 x 10 in., illus., cloth. $10.00.

A directory of producers, manufacturers, dealers and consumers connected with the architectural, contracting, electrical, engineering, hardware, iron, mechanical, mill, mining, quarrying, railroad, steel and kindred industries.

HEATON'S ANNUAL;

The Commercial Handbook of Canada and Boards of Trade Register, 13th Year, 1917. Toronto, Heaton's Agency, 1917. 518 pp., 5 x 7 in., cloth. $1.25.

Collects in one volume the information of value to merchants and manufacturers. Includes lists of government officials, customs brokers, banking towns, registration offices, shipping directions, etc. The customs tariff and a digest of customs law and regulation are given. The work also contains much general information on the resources of the country, a gazetteer of commercial towns and an economic bibliography of governmental reports, together with the usual tables.

THE WOOL INDUSTRY;

Commercial Problems of the American Woolen and Worsted Manufacture. By Paul T. Cherington. N. Y., Chic., Lond., A. W. Shaw Co. (copyright 1916). 16 + 261 pp., 5 x 8 in., cloth. $2.50.

The author's purpose has been to present the results of an examination of the industries producing woolen and worsted fabrics, approached from the side of their buying and selling problems. Omits other features of the industry, such as wool-growing, tariff relations, manufacturing technique, etc.

A DISCUSSION OF THE PRINCIPLES AND PRACTICE UNDERLYING CHARGES FOR WATER, GAS, ELECTRICITY, COMMUNICATION AND TRANSPORTATION SERVICES.

By Harry Barker. N. Y., McGraw-Hill Book Co., Inc.; Lond., Hill Publishing Co., Ltd., 1917. 14 + 387 pp., 9 x 6 in., cloth. $4.00.

The author's aim has been to present, concisely and impartially, and as far as possible, in non-technical language, the diverse phases of rate-making for public utilities, including a comprehensive discussion of (1) such corporation and municipal activities as affect service and rates; (2) the trend of public opinion and Court and commission decisions; and (3) the most important engineering and economic problems involved, with the hope that it will prove of service to lawyers and legislators, to editorial writers of the daily press, to students of municipal affairs, and to the general public. Contents: Development of Utility Regulation, Utility Privileges and Obligations, Rights of the Public; Product and Service Companies, Some Definitions of Rates and Services; Various Bases for Rates; Details of the Cost-of-Service Study of Rates, Test for Fixed and Operating Charges; Fair Value of a Utility Property; Valuation as an Engineering Task, Appraisal of Land and Water Rights; Reasonable Return, Interest, Compensation for Risk and Attention, Extra Profits; Depreciation as it Affects Utility Rates; Miscellaneous Problems Indirectly Related to Rate-Making; Problems of Railway Rates; Problems of Express Transportation Rates; Rate Problems of Street and Interurban Railway Transportation; Problems of Water Rates; Rate Problems of Gas Utilities; Rate Problems of Electricity Supply Works; Problems of Telephone Rate-Making; Appendix A. B. C. D.

VALUATION, DEPRECIATION AND THE RATE-BASE.

By Carl Ewald Grunsky and Carl Ewald Grunsky, Jr. N. Y., John Wiley & Sons, Inc.; Lond., Chapman & Hall, Ltd., 1917. 8 + 387 pp., 9 x 6 in., cloth. $4.00. (Gift of the author and publisher.)

The author states that this book is the result of personal contact with the valuation problem. Special consideration has been given to a discussion of the non-agreement of the actual life of articles which have a limited period of usefulness with their probable or normal life. The tables included are not only intended for valuation engineers, but for any one having problems of finance and bonding to solve. Contents: Introduction and General Notes; Definitions; Fundamental Principles; Essentials of Value; Elements Which Reduce Value; The Effect of the Non-Agreement Between Actual and Probable Life Upon the Determination of Depreciation; The Purpose of the Appraisal; The Fixing of Rates; Possible Procedures when the Rates for a Public Service are to be Fixed; Notes on the Determination of the Value of Real Estate in Eminent Domain Proceedings and for Rate Fixing Purposes; The Value of a Water-Right and of Reservoir and Watershed Lands; The Accounting System; The Valuation of Mines and Oil Properties by C. E. Grunsky, Jr ; Tables.

THE TAYLOR SYSTEM OF SCIENTIFIC MANAGEMENT.

By C. Bertrand Thompson. N. Y., Chic., Lond., A. W. Shaw Co. (copyright 1917). 175 pp., 22 illus., 5 diagrams, 9 x 11 in., flexible cloth. $10.00.

Intended to give, in addition to the history and theory of the system, enough development and operation to enable the factory manager to visualize the system in some detail, to distinguish clearly between it and other systems, and to understand its principles and mechanisms as found in actual practice. Based on personal investigations of all the installations of the system between Maine, Maryland and Chicago. Contains a bibliography of the important publications on the system.

INDUSTRIAL PREPAREDNESS.

By C. E. Knoeppel. N. Y., The Engineering Magazine Co., 1916. 6 + 145 pp., 8 x 5 in., cloth. $1.00.

A study of Germany's military and industrial preparedness intended to point the way to national greatness through the right kind of social, industrial and military preparedness.

HOW TO FIND FACTORY COSTS:

By C. Bertrand Thompson. Chic., N. Y., Lond., A. W. Shaw Co. 191 pp., 10 x 7 in., 51 charts, 1 diagram, 1 tab., cloth. $3.00.

It is stated that this book is broad enough to apply to all kinds of industries, and is intended to be useful to the accountant as well as the factory head.

WORKMEN'S COMPENSATION LAW:

Personal Injury by Accident Arising Out of and in the Course of the Employment. By P. Tecumseh Sherman. N. Y., Workmen's Compensation Publicity Bureau (copyright 1916). 67 pp., 6 x 9 in., paper. $2.00.

A compilation of the decisions construing the British law on the subject, with abbreviated summaries of the relevant portions of the French and German laws. These precedents will be useful, the author believes, in defining the meaning of "accidents due to risk of work" as used in the American statutes.

THE ESSENTIALS OF AMERICAN TIMBER LAW.

By J. P. Kinney. N. Y., John Wiley & Sons, Inc.; Lond., Chapman & Hall, Ltd., 1917. 19 + 279 + 10 pp. 6 x 9 in., cloth. $3.00.

A presentation of the existing law regarding trees and their products as property, with such observations and references to historical development as are considered necessary to an understanding of the reasons for the law. Citations to the sections of the compiled or session laws of the different States enables the reader to find the various statutes in full. Especial prominence is given to interpretations of the law by the Courts.

THE THEORY AND PRACTICE OF WORKING PLANS.

(Forest Organization). By A. B. Recknagel. 2d ed., rev. N. Y., John Wiley & Sons; Lond., Chapman & Hall, Ltd., 1917. 14 + 265 pp., 14 illus., 6 pl., 1 tab., 6 x 9 in., cloth. $2.00.

An adaptation of the best European efforts in forest organization to the present needs of American forestry, intended for the practising forester as well as the student. The nomenclature of this edition has been revised in accordance with the suggestions of the Committee on Terminology of the Society of American Foresters, and the text as a whole has been revised and extended.

ANNUAL CHEMICAL DIRECTORY OF THE UNITED STATES.

B. F. Lovelace, Editor. Baltimore, Williams & Wilkins Co. (copyright 1917). 305 pp., 6 x 9 in., cloth. $5.00.

First issue. Includes American manufacturers of and dealers in chemical apparatus and equipment ; and professional chemical firms and laboratories. Lists of colleges offering instruction in chemistry, experiment stations, technical and scientific societies are given, also of Federal and State officials of dairying, foods, drugs, etc. Bibliographies of American and foreign journals and important books of the year, with a concise review of new happenings, devices, methods and appliances conclude the book.

HANDBOOK OF CHEMISTRY AND PHYSICS:

A Ready-Reference Pocket Book of Chemical and Physical Data. 5th ed. Cleveland, The Chemical Rubber Co., 1917. 414 pp., 4 x 7 in., cloth. $2.00.

The present edition of this convenient reference book has been carefully revised, many new tables have been added, and the index has been enlarged.

A TEXT-BOOK OF INORGANIC CHEMISTRY.

Edited by J. Newton Friend. Vol. VIII: The Halogens and Their Allies. By Geoffrey Martin and E. A. Dancaster. Lond., Charles Griffin & Co., Ltd., 1915. 18 + 337 pp., 30 illus., 1 tab., 6 x 9 in., cloth. $3.00. (Gift of J. B. Lippincott.)

A concise general account of the chief chemical and physical properties of fluorine, chlorine, bromine, iodine, and manganese, and their compounds. Describes the most important manufacturing operations briefly. The work does not attempt to be exhaustive, but is provided with very numerous references to original publications on the various phases of the subjects dealt with.

ELEMENTARY CHEMICAL MICROSCOPY.

By Emile Monnin Chamot. N. Y., John Wiley & Sons, Inc.; Lond., Chapman & Hall, Ltd., 1916. 13 + 410 pp., 147 illus., 6 x 9 in., cloth. $3.00.

The author believes that the American chemist has failed to appreciate the benefits to be obtained by the application of chemical microscopic methods in the industries and in research. This book is written to instruct him in the manipulation of the microscope and in the great variety of problems which may be solved by its use, and to call attention to the fields in which it is especially useful.

OUTLINES OF INDUSTRIAL CHEMISTRY:

A Text-Book for Students. By Frank Hall Thorp, with Assistance in Revision from Warren K. Lewis. 3d ed. rev. and enl. N. Y., The Macmillan Co., 1916. 25 + 665 pp., 137 illus., 6 x 9 in., cloth. $3.75.

The present edition of this well-known treatise has been thoroughly revised, many sections having been rewritten and much obsolete matter replaced by new material. The modern concepts and theories of chemistry have been introduced whenever these promised to make clearer the phenomena involved.

GENERAL INSTRUCTIONS AND METHODS OF ANALYSIS AND CHEMICAL CONTROL FOR USE IN THE FACTORIES OF THE CUBAN-AMERICAN SUGAR COMPANY.

By Guilford L. Spencer. N. Y., The Cuban American Sugar Company, 1916. 39 pp., 8½ x 6 in., paper. $1.00.

The purpose of this book is to supply methods that are applicable in all the company's factories, in obtaining the data required for report sheets and permanent records. Definitions of terms used by the company are included.

A HANDBOOK FOR CANE-SUGAR MANUFACTURERS AND THEIR CHEMISTS.

By Guilford L. Spencer. 5th ed. enl. N. Y., John Wiley & Sons, Inc.; Lond., Chapman & Hall, Ltd., 1916. 15 + 529 pp., 7 x 4 in., 83 illus., 1 pl., leather. $3.50.

The book is divided into two parts; Manufacture of Cane-Sugar, and Sugar Analysis; a chapter is also included on sugar refining as practised in the United States. The chemical section of the book has been revised to meet the conditions of the very large factories now in operation.

THE LIFE OF ROBERT HARE:

An American Chemist (1781-1858). By Robert Fahs Smith, Provost of the University of Pennsylvania. Phila. and Lond., J. B. Lippincott Co., 1917. 508 pp., 1 por., 4 illus., 6 x 9 in., cloth. $5.00.

Assembles the labors of Robert Hare in such a form as to acquaint students of chemistry with him, and to show his title to an exalted place in the history of chemistry in this country. Told largely by hitherto unpublished letters and documents collected from forgotten journals and pamphlets.

HOW TO STUDY.

By George Fillmore Swain. N. Y., McGraw-Hill Book Co., Inc.; Lond., Hill Publishing Co., Ltd., 1917. 10 + 65 pp., 5 x 7 in., paper. 25 cents.

Dr. Swain's little monograph is intended to formulate, briefly and simply, certain fundamental principles of study and to point the way to proper habits and methods. Intended for students in and graduates of technical schools. Contents: Introduction; The Proper Mental Attitude; Studying Understandingly; System; Mental Initiative; Habits of Work; Suggestions to Teachers.

ARITHMETIC FOR ENGINEERS;

Including Simple Algebra, Mensuration, Logarithms, Graphs, and the Slide Rule. By Charles B. Clapham. (The D. U. Technical Series.) N. Y., E. P. Dutton & Co. (preface 1916). 436 pp., 149 illus., 6 x 9 in., cloth. $3.00.

The author's object is to treat the subject with sufficient detail and enough engineering application to provide a truly practical treatise, omitting all subjects which have only academic interest.

APPLIED MECHANICS.

By Alfred Poorman. N. Y., McGraw-Hill Book Co., Inc.; Lond., Hill Publishing Co., Ltd., 1917. 11 + 244 pp., 321 illus., 6 x 9 in., cloth. $2.00.

A text-book for undergraduate courses in engineering schools, in which the basic principles have been developed in a way which the average student can follow easily and illustrations showing clearly the application of such principles to the solution of engineering problems have been provided. Extended use has been made of the graphic method of solution.

LAWS OF PHYSICAL SCIENCE.

A Reference Book. By Edwin F. Northrup. Phila. and Lond., J. B. Lippincott Co., 1917. 7 + 210 pp., 5 x 8 in., leather. $2.00.

A summary of the general propositions underlying physical science and engineering, classified for convenience in consultation. Each law is accompanied by one or more references to more extended information. A bibliography is included.

FRENCH MEASURE AND ENGLISH EQUIVALENTS.

By John Brook. N. Y., Spon & Chamberlain; Lond., E. & F. N. Spon, 1917. 7 + 80 pp., 4 x 3 in. 40 cents.

Vest-pocket size. Gives the English equivalents of the metric measures of length and weight and of the old French, Prussian, Austrian, and Russian measures of length. A table for reducing the usual vulgar fractions of the inch to decimals is also given. The English equivalents are given in decimal fractions, correct to six places, and approximately in vulgar fractions as well. Intended for engineers, manufacturers, and workmen.

MUNICIPAL ENGINEERING PRACTICE.

By A. Prescott Folwell. N. Y., John Wiley & Sons, Inc.; Lond., Chapman & Hall, Ltd., 1916. 11 + 422 pp., 9 x 6 in., 112 illus., 1 pl., cloth. $3.50.

This book has been written to treat at greater length practical information concerning street cleaning, public comfort stations and other subjects not treated in most of the text-books on municipal engineering. Contents: Fundamental Data; The City Plan; Street Surface Details; Bridges and Waterways; City Surveying; Street Lights, Signs and Numbers; Street Cleaning and Sprinkling; Disposing of City Wastes; Markets, Comfort Stations and Baths; Parks, Cemeteries and Shade Trees.

MUNICIPAL ACCOUNTING.

By DeWitt Carl Eggleston. N. Y., The Ronald Press Co., 1914. 22 + 456 pp., 9 x 6 in., 113 diagrams, ½ leather. $4.00.

Intended to furnish practising accountants, municipal accountants and students with a complete method of municipal accounting, representing the best modern practice. The methods described are intended for the larger cities, but a special chapter on accounting for smaller cities is included. Many of the methods used in New York are presented.

INSTRUCTIONS TO LOCATING ENGINEERS AND FIELD PARTIES.

By F. Lavis. N. Y., McGraw-Hill Book Co., Inc. (copyright 1916). 44 pp., 9 x 6 in., 9 diagrams, cloth. $1.00. (Gift of the author.)

Reproduced in abridged form from "Railroad Location Surveys and Estimates." Originally prepared for the use of field parties working under his direction in the United States, afterwards modified for use in South America, and finally prepared in the present form for use in China. They are intended to secure uniform practice in making surveys and in the compilation of the results in the form of maps, estimates, etc.

WINTER TRACK WORK.

By E. R. Lewis. Chic., Railway Educational Press, Inc. (copyright 1917). 166 pp., 21 illus., 5 x 8 in., cloth. $1.60.

Explains practical methods for maintaining railway track. Intended for use of trackmen. Contents: Climate and Track; Frost; Snow; Shims and Shimming; Winter Track Force, Tools and Supplies; Snow Fences and Snow Sheds; Snow Handling Equipment; Spring Floods; Storing Ice; Organization.

RAILWAY TRACK ECONOMICS:

A Tabloid Treatise Upon Railroad Problems. By August E. Liebmann. rev. ed. Chic. (privately printed), 1916. 66 pp., 6½ x 5 in., leather. $3.00. (Gift of the author.)

The author states that he presents in tabloid form some criticism and recommendations for solving the problems of maintenance costs.

ELECTRIC TRACTION:

A Treatise on the Application of Electric Power to Tramways and Railways. By A. T. Dover. N. Y., and Lond., Whittaker & Co., 1917. 18 + 667 pp., 9 x 6 in., 518 illus., 5 pl., cloth.

Intended for engineers and advanced students. Representative examples of modern tramway and railway practice are included, but detailed accounts of electrification have been omitted and generating stations and transmission lines have not been considered. Contains a bibliography on electrification, and a number of worked examples have been included in the text. Contents: Mechanics of Train Movements; Motors; Control; Auxiliary Apparatus; Rolling Stock; Detailed Study of Train Movement; Track and Overhead Construction; Distributing Systems and Sub-stations.

ELECTRIC RAILWAY TRANSPORTATION.

By Henry W. Blake and Walter Jackson. N. Y., The McGraw-Hill Book Co., Inc.; Lond., Hill Publishing Co., Ltd., 1917. 7 + 487 pp., 120 illus., 6 x 9 in., cloth. $5.00.

The first book, the authors state, devoted to transportation methods and practice. Largely compiled to put in convenient form matter which has appeared in the *Transactions* of the American Electric Railway Transportation and Traffic Association and in the electric railway periodicals. Contents: Organization and Definitions; Adjustment of Service to Traffic; Accelerating Traffic Movement Along the Line; Accelerating Traffic Movement on the Car; Car Types in Relation to Traffic; City Timetables-Preliminaries; Interurban Schedules and Dispatching; Fares; Fare Collection Practices and Devices; Public Relations; Promotion of Passenger Traffic; Traffic Signs for Cars, Station and Road-Information for the Public; Competition; Freight and Express Business; Selection and Training of Men; Wages and Wage Agreements; Welfare Work; Discipline of Trainmen; Forms of Extra Pay.

INTERBOROUGH FINANCE:

Present and Future with Especial Reference to Conditions When the New Lines shall have been Completed. Including Synopsis of the Financial Structure of Interborough Consolidated Corporation. N. Y., Van Emburgh & Atterbury, 1917. 69 pp., 2 maps, 2 diagrams, 6 x 9 in., leather.

Contents: Interborough Consolidated Corporation; Interborough Rapid Transit Company; New York Railways Company; Partnership Between Interborough and City of New York; Will the Interborough Earn its Preferentials.

LABORATORY MANUAL OF BITUMINOUS MATERIALS:

For the Use of Students in Highway Engineering. By Prevost Hubbard. N. Y., John Wiley & Sons; Lond., Chapman & Hall, Ltd., 1916. 11 + 153 pp., 9 x 6 in., 38 illus., cloth. $1.50.

Certain fundamentals not strictly a part of laboratory work have been included as a guide to students not well versed in the nomenclature, classification, and uses of bituminous materials.

A TEXT-BOOK ON BRICK PAVEMENTS:

By Clark R. Mandigo. Kansas City, Western Paving Brick Manufacturers Association, 1917. 126 pp., 42 illus., 5 x 8 in., cloth. $1.50.

Intended for municipal officers, commissioners and citizens as well as for city and county engineers and contractors. Descriptive rather than technical, but intended to give accurate information of interest to all concerned in road improvement. Contents: Brick Pavements; Highway Economics; The Sub-Grade and the Foundation; Manufacture of Brick; Paving Problems; Appendix: Standard Specifications.

THE VENTILATION HANDBOOK:

The Principles and Practice of Ventilation as Applied to Furnace Heating; Ducts, Flues and Dampers for Gravity Heating; Fans and

Fan Work for Ventilation and Hot Blast Heating. By Charles L. Hubbard. N. Y., Sheet Metal Publishing Co., 1916. 218 pp., 8 x 6 in., 137 illus., cloth. $2.00.

A series of questions, answers and descriptions, with illustrations, arranged from a series of articles prepared for *Sheet Metal*. Care has been taken to keep all descriptions and mathematical work well within the understanding of the student and beginner.

PRACTICAL SHEET METAL DUCT CONSTRUCTION:

A Treatise in the Construction and Erection of Heating and Ventilating Ducts. By William Neubecker. N. Y., The Sheet Metal Publishing Co., 1916. 194 pp., 8 x 6 in., 217 illus., cloth. $2.00.

The plan of the present work is to take up each operation, and by means of descriptions (usually illustrated) to show all operations incident to the construction and erection of heating and ventilating ducts.

GRAY'S PLUMBING DESIGN AND INSTALLATION.

By William Beall Gray. N. Y., David Williams Co., 1916. 559 pp., 500 illus., 6 x 9 in., cloth. $4.00. (Gift of U. P. C. Book Co.)

A reference work for plumbers, intended to illustrate fully standard American practice in all branches of the plumbing and allied trades.

STATE SANITATION:

A Review of the Work of the Massachusetts State Board of Health. By George Chandler Whipple. Cambridge, Harvard University Press; Lond., Oxford University Press, 1917. 11 + 377 pp., 12 pl., 8 diagrams, 6 x 9 in., cloth. $2.50.

The first of a three-volume history; intended to set forth the past work of the Board, to reprint selected articles of importance from the older reports and abstracts of the others, and to serve as an index and guide to the annual reports. Biographical sketches of the engineers, chemists, and biologists of the Board will be included. Volume one contains the history of the Board since its establishment in 1869, together with an abridged version of the "Report of the Massachusetts Sanitary Commission of 1850."

A MANUAL OF FIRE PREVENTION AND FIRE PROTECTION FOR HOSPITALS.

By Otto R. Eichel. N. Y., John Wiley & Sons; Lond., Chapman & Hall, Ltd., 1916. 5 + 69 pp., 7½ x 5 in., cloth. $1.00.

An outline of the principles of fire prevention and protection, with indications for their application in institutions housing the sick, based on the personal observation and study of the author, who is Director of the Division of Sanitary Supervisors, New York State Department of Health.

FIRE PREVENTION AND PROTECTION;

A Compilation of Insurance Regulations Covering Modern Restrictions on Hazards and Suggested Improvements in Building Construction and Fire Prevention and Extinguishment. By A. C. Hutson, Fire Protection Engineer. 3d ed. completely rev. N. Y. and Chic., The Spectator Co., 1916. 7 + 777 pp., 126 illus., 3 pl., 5 x 7 in., leather. $4.25.

Written especially for merchants, manufacturers and underwriters, as a succinct presentation of the knowledge necessary to attain as complete protection as possible from fire. Covers all the suggested regulations of the National Board of Fire Underwriters and allied organizations.

ELLIOTT'S WEIGHTS OF STEEL;

For Engineers, Architects, Contractors, Builders, Steel Manufacturers, and all Users of Rolled Steel. Computed by Thomas J. Elliott. Cleveland, The Penton Publishing Co., 1916. 662 pp., 6 x 9 in., leather. $20.00.

Given the weight of a linear foot of any section of rolled steel, this book of tables enables one to find the weight of a single piece of any length without performing the operation of multiplication.

BUILDING SUPERINTENDENCE FOR STEEL STRUCTURES;

A Practical Work on the Duties of a Building Superintendent for Steel-Frame Buildings and the Proper Methods of Handling the Materials and Construction. By Edgar S. Belden. Chic., American Technical Society, 1917. 95 pp., 25 illus., 2 pl., 5 x 8 in., cloth. $1.00.

Concise practical discussion of the problems which confront the superintendent of structural steel construction, and of the proper methods of meeting them.

STRESSES IN STRUCTURES.

By A. H. Heller. Rev. by Clyde T. Morris. 3d ed. N. Y., John Wiley & Sons, Inc.; Lond., Chapman & Hall, Ltd., 1916. 13 + 374 pp., 230 illus., 6 x 9 in., cloth. $2.75.

Author has attempted to provide a book which, without being an exhaustive scientific treatise on stresses, would be a suitable text-book for students and also a concise reference book for engineers. In the present edition various explanations have been expanded, numerical illustrative examples have been added and new material introduced where the reviser has found it necessary.

MODERN UNDERPINNING:

Development, Methods and Typical Examples. By Lazarus White and Edmund Astley Prentis, Jr. (Wiley Engineering Series No. 2.) N. Y., John Wiley & Sons, Inc.; Lond., Chapman & Hall, Ltd., 1917. 12 + 94 pp., 9 x 6 in., 48 illus., 1 pl., cloth. $1.50.

Intended to exhibit, by means of photographs and drawings, the essential steps in underpinning as illustrated by the methods used in subway construction in New York City. Only enough text to supplement the illustrations included. Contents: General Aspects; Development of Underpinning and Methods; Shores, Needles, and Foundation Reinforcements; Specific Examples of Underpinning; Appendix.

THE INDUSTRIAL AND ARTISTIC TECHNOLOGY OF PAINT AND VARNISH.

By Alvah Horton Sabin. 2d ed. rev. N. Y., John Wiley & Sons, Inc.; Lond., Chapman & Hall, Ltd., 1917. 10 + 473 pp., 9 x 6 in., 10 pl., 10 illus., cloth. $3.50.

Written to give a correct general outline of the subject, with a brief account of the modern use of paints and varnishes, and of the principles involved in their fabrication and application. This edition is nearly one-third larger than the first one and takes cognizance of the changes in the character of the cheaper varnishes due to the use of tung oil.

ARCHITECTURAL DRAWING AND LETTERING;

A Manual of Practical Instruction in the Art of Drafting and Lettering for Architectural Purposes, Including the Principles of Shading and Rendering and Practical Exercises in Design. Part 1, Architectural Drawing. By Frank A. Bourne and H. V. Von Holst. Part 2, Architectural Lettering. By Frank Choteau Brown. Chic., American Technical Society, 1917. 102 + 48 pp., 94 illus., 6 x 8 in., cloth. $1.50.

This is an elementary work presenting the art in logical manner.

MILITARY SKETCHING AND MAP READING.

By Capt. Loren C. Grieves. Wash., U. S. Infantry Assoc., 1917. 95 pp., 32 illus., 6 x 9 in., cloth. $1.00.

A text-book intended for officers of the National Guard, candidates for commissions in the Army and the Reserve Officers' Training Corps and for educational institutions. Meets the provisions prescribed by the War Department.

TOPOGRAPHICAL DRAWING.

By Edwin R. Stuart. N. Y., McGraw-Hill Book Co., Inc.; Lond., Hill Publishing Co., Ltd., 1917. 126 pp., 54 illus., 1 map, 6 x 9 in., cloth. $2.00.

Intended to furnish a satisfactory standard of practice in topographical drawing which will combine good execution with economy of time. Contents: Introductory; Map Projection; Instruments and Drawing Materials; Plotting; Special Methods in Free-Hand Drawing; Practice in Topographical Drawing; Lettering; Conventional Signs; Map Drawing.

THE BAROMETRICAL DETERMINATION OF HEIGHTS:

A Practical Method of Barometrical Levelling and Hypsometry for Surveyors and Mountain Climbers. By F. J. B. Cordeiro. 2d ed. rev. and enl. N. Y., Spon & Chamberlain; Lond., E. & F. N. Spon, Ltd., 1917. 26 pp., 7 x 4 in., cloth. 50 cents.

This little volume was an essay originally entered in the Hodgkins Prize Competition held under the auspices of the Smithsonian Institution, and was awarded honorable mention.

AZIMUTH.

By George L. Hosmer. 2d ed. rev. N. Y., John Wiley & Sons, Inc.; Lond., Chapman & Hall, Ltd., 1916. 5 + 73 pp., 5 x 7 in., 6 illus., leather. $1.00.

This handbook is intended to present in compact form certain approximate methods of determining the true bearing of a line, together with the necessary rules and tables arranged in a simple manner so that they will be useful to the practical surveyor. This edition contains a new method for finding the azimuth by an observation on the pole star at any hour angle when the local time is known. The tables of the sun's declination have been extended to 1919, and new star maps are given.

STANDARD METHODS FOR THE EXAMINATION OF WATER AND SEWAGE.

By the American Public Health Association. 3d ed. Bost., American Public Health Association, 1917. 115 pp., 10 x 7 in., 1 illus., cloth. $1.25.

This volume is the result of the combined labors of three committees namely: the Committees of the American Public Health Association; American Chemical Society; and the Association of Official Agricultural Chemists. The 1912 edition has been modified by the addition of methods for the examination of sewage sludge and muds, the analysis of chemicals used in the treatment of water, and the determination of chlorine. Changes also have been made in the technique of existing methods. Bacteriological, chemical and microscopical bibliographies are included.

WATER PURIFICATION.

By Joseph W. Ellms. N. Y., McGraw-Hill Book Co., Inc.; Lond., Hill Publishing Co., Ltd., 1917. 10 + 485 pp., 149 illus., 6 x 9 in., cloth. $5.00.

Intended as a fairly complete account of the development of the art. Includes a consideration of the properties of various classes of waters and gives especial attention to the relation of polluted public water supplies to water-borne diseases. Describes in considerable detail the various steps in purification processes, such as

sedimentation, coagulation, filtration and disenfection. Contains chapters on water softening and on the removal of iron and manganese from ground-water supplies. Bibliographical references accompany each chapter.

USE OF WATER IN IRRIGATION.

By Samuel Fortier. 2d ed. N. Y., McGraw-Hill Book Co., Inc.; Lond., Hill Publishing Co., Ltd., 1916. 16 + 325 pp., 8 x 6 in., 75 illus., 8 pl., cloth. $2.00.

The first edition, which appeared in 1914, dealt with the agricultural side of irrigation. In the present edition the article on the measurement of water has been revised and enlarged, and a new one added on sewage irrigation. A new chapter on the "Use of Water in Foreign Countries" is a most important addition to the book. Contents: Introduction; The Irrigated Farm; The Necessary Equipment and Structures; Methods of Preparing Land and Applying Water; Waste Measurement; Delivery and Duty of Water; Irrigation of Staple Crops; Use of Water in Foreign Countries.

IRRIGATION WORKS CONSTRUCTED BY THE UNITED STATES GOVERNMENT.

By Arthur Powell Davis. N. Y., John Wiley & Sons, Inc.; Lond., Chapman & Hall, Ltd., 1917. 16 + 413 pp., 128 illus., 6 x 9 in., cloth. $4.50.

Contains engineering descriptions, with the necessary illustrations, of the various projects undertaken by the Reclamation Service. The projects described are the Salt River, the Yuma, the Orland, the Grand Valley, the Uncompahgre, the Boise, the Minidoka, the Huntley, the Lower Yellowstone, the North Platte, the Truckee-Carson, the Carlsbad, the Hondo, the Rio Grande, the Umatilla, the Klamath, the Belle Fourche, the Strawberry Valley, the Okanogan, the Yakima and the Shoshone. Intended for engineers and those interested in the development of arid lands.

RIVER DISCHARGE.

By John Clayton Hoyt and Nathan Clifford Grover. 4th ed. rev. and enl. N. Y., John Wiley & Sons, Inc.; Lond., Chapman & Hall, Ltd., 1916. 12 + 210 pp., 16 x 9 in., 23 illus., 7 pl., 2 maps, 1 diagram, tab., cloth. $2.00.

This edition has been revised and brought up to date. Chapter V, "Discussion and Use of Data" has been largely rewritten, and Chapter VI has been expanded to cover the field of hydrology. Contents: Introduction; Instruments and Equipment; Structures from Which Measurements are Made; Velocity-Area-Stations; Weir Stations; Discussion and Use of Data; Hydrology as Related to Stream Flow.

THE PANAMA CANAL AND COMMERCE.

By Emory R. Johnson. N. Y. and Lond., D. Appleton & Co., 1916. 295 pp., 8 illus., 7 maps, 5 x 7 in., cloth. $2.00.

Explains the reasons for the building of the canal and discusses the use of the waterway by the commerce and shipping of the United States and other countries. Intended for those engaged in shipping and other students of the canal in relation to commerce. A companion volume to Gorgas' "Sanitation in Panama" and Sibert and Stevens' "The Construction of the Canal."

MEMBERSHIP

(From May 4th to August 2d, 1917)

ADDITIONS

MEMBERS		Date of Membership.
ALLISON, JOSEPH CHESTER. Cons. Engr., 209 Anderson Bldg., Calexico, Cal.	Assoc. M. M.	Sept. 3, 1912 June 12, 1917
AYRES, LYONEL. Engr., City of Duluth, City Hall, Duluth, Minn.		June 11, 1917
BAKHMETEFF, BORIS ALEXANDROWITCH. 1014 Flatiron Bldg., New York City		Mar. 13, 1917
BARTLETT, CHARLES TERRELL. Cons. Engr. (Bartlett & Ranney, Inc.), 108 East Crockett St., San Antonio, Tex.	Assoc. M. M.	Jan. 2, 1912 July 12, 1917
BEGIEN, RALPH NORMAN. Gen. Mgr., B. & O. R. R., Room 507, B. & O. Bldg., Baltimore, Md.	Assoc. M. M.	June 7, 1905 Mar. 13, 1917
BENEDICT, FARRAND NORTHROP. Vice-Pres. and Engr., Thomas Crimmins Contr. Co.; Res., 33 South Maple Ave., East Orange, N. J.	Jun. Assoc. M. M.	June 4, 1907 May 2, 1911 May 15, 1917
BLACK, ERNEST BATEMAN. Cons. Engr. (Black & Veatch), 507 Inter-State Bldg., Kansas City, Mo.	Assoc. M. M.	Nov. 1, 1910 April 18, 1917
BLAIR, CLARENCE MOORE. 785 Edgewood Ave., New Haven, Conn.	Assoc. M. M.	Dec. 5, 1911 May 15, 1917
BOWIE, ALFRED WILLIAM. Engr. in Chg., Westinghouse, Church, Kerr & Co., 37 Wall St., New York City.		June 11, 1917
BRAUNE, GUSTAVE MAURICE. Associate Prof., Civ. Eng., Univ. of Cincinnati, 248 Loraine Ave., Cincinnati, Ohio	Jun. Assoc. M. M.	June 2, 1896 Sept. 4, 1901 May 15, 1917
BRAUNWORTH, PERCY LEWIS. Borough Engr. of Roseland; Contr. Engr., 36 Spring St., Montclair, N. J.	Assoc. M. M.	Jan. 3, 1911 June 12, 1917
CAHILL, JOHN RICHARD. Gen. Contr., 110 Sutter St., San Francisco, Cal.	Jun. Assoc. M. M.	April 30, 1907 June 4, 1913 June 12, 1917
CARSTARPHEN, FREDERICK CHARLES. Tramway Engr., Am. Steel & Wire Co., Trenton, N. J.	Assoc. M. M.	April 6, 1909 May 15, 1917
CORTRIGHT, EDWIN KEEN. Res. Engr., Lawrence Central Bridge, 40 Lawrence St., Lawrence, Mass.	Jun. Assoc. M. M.	Feb. 4, 1908 April 30, 1912 June 12, 1917
CROMWELL, GEORGE. City Engr., 4318 Sierra Vista St., San Diego, Cal.	Assoc. M. M.	Oct. 3, 1911 June 12, 1917
CUDEBEC, ALBERT BENNETT. Special Eng. Investigator, Elec. Bond & Share Co., 71 Broadway, New York City.		May 15, 1917

MEMBERS (*Continued*)

Name and Address	Grade	Date of Membership.
CURD, WILLIAM CANTRILL. Contr. Engr., R. R. Dept., Layne & Bowler Co., Randolph Bldg., Memphis, Tenn.		June 11, 1917
DAVIES, JOHN PERCIVAL. Office Engr., Honolulu Iron Works Co., Woolworth Bldg., New York City	Assoc. M. M.	May 2, 1911 May 15, 1917
DEWELL, HENRY DIEVENDORF. 58 Sutter St., San Francisco, Cal	Assoc. M. M.	May 2, 1911 June 12, 1917
DICKINSON, WILLIAM DEWOODY. Cons. Engr. (Dickinson & Watkins), 610 State Bank Bldg., Little Rock, Ark	Assoc. M. M.	May 3, 1910 June 12, 1917
ELAM, WILLIAM EARLE. Asst. Engr., Mississippi Levee Board, Greenville, Miss	Jun. Assoc. M. M.	Feb. 6, 1906 Oct. 31, 1911 June 12, 1917
GARDNER, HARRY CARTER. Chf. Engr., John H. Wickersham, 724 North Lime St., Lancaster, Pa	Jun. Assoc. M. M.	July 1, 1909 Nov. 12, 1913 June 12, 1917
GAUMER, ALBERT WESLEY. Supt., Juragua Iron Co., Box 383, Santiago de Cuba, Cuba	Assoc. M. M.	May 31, 1910 June 12, 1917
GENDELL, DAVID SMITH, JR. Mgr. of Erection, McClintic-Marshall Co., Pottstown, Pa		June 11, 1917
GENTNER, OTTO HENRY, JR. Cons. Engr., Northwest Cor., 16th and Sansom Sts., Philadelphia, Pa	Assoc. M. M.	Nov. 4, 1908 May 15, 1917
GEORGE, HOWARD HOWELL. Asst. Engr., M. of W., Public Service Ry., Room 646, Public Service Terminal Bldg., Newark, N. J	Assoc. M. M.	April 1, 1914 June 12, 1917
GINSBURG, SAMUEL ROWLAND. Gen. Supt., Central Romana, La Romana, Dominican Republic	Assoc. M. M.	May 28, 1912 May 15, 1917
GREEN, CHARLES NEWTON. Engr., Subsurface Structures, Public Service Comm., First Dist., 120 Broadway, New York City	Assoc. M. M.	June 3, 1903 June 12, 1917
GRONER, TRYGVE DANIEL BODTKER. Chf. Engr., D. T. & I. R. R., Springfield, Ohio	Assoc. M. M.	Mar. 4, 1914 June 12, 1917
GUTTIN, HENRY. Mech. Engr., Am. Cotton Oil Co., 27 Beaver St., New York City		May 15, 1917
HENDERSON, SAMUEL WHILDEN. Gen. Mgr., Marysville Light, Power & Water Co.; Vice-Pres. and Gen. Mgr., The Excelsior Springs Water, Gas & Elec. Co., Excelsior Springs, Mo	Assoc. M. M.	Sept. 6, 1905 June 12, 1917
HIGGINS, HERMAN KEENE. 209 McBride St., Jackson, Mich	Assoc. M. M.	Nov. 7, 1906 Jan. 16, 1917

MEMBERS (*Continued*)

		Date of Membership.
HOWARD, OLIVER ZELL. Care, The Diamond Match Co., Lawrence, Mass	Assoc. M. M.	Feb. 28, 1911 May 15, 1917
HOWE, CLARENCE DECATUR. Cons. Engr., 2d Floor, Whalen Bldg., Port Arthur, Ont., Canada	Jun. Assoc. M. M.	Oct. 5, 1909 May 7, 1913 June 12, 1917
HUBER, WALTER LEROY. Cons. Engr., 1304 First National Bank Bldg., San Francisco, Cal	Jun. Assoc. M. M.	April 3, 1906 Mar. 1, 1910 June 12, 1917
HURLBUT, HINMAN BARRETT. Cons. Engr., 3318 Nineteenth St., N. W., Washington, D. C.	Assoc. M. M.	July 1, 1909 June 12, 1917
LUDLOW, JUSTIN WYMAN. Asst. Engr., Los Angeles Harbor Dept., City Hall, San Pedro, Cal	Jun. Assoc. M. M.	April 5, 1904 Mar. 1, 1910 Jan. 16, 1917
LUND, ALFRED MAJENDIE. 2618 Gaines St., Little Rock, Ark	Assoc. M. M.	April 30, 1912 April 18, 1917
LYNDON, LAMAR. Cons. Engr. (Lyndon & Taylor), 21 Park Row, New York City		May 15, 1917
MACKALL, JOHN NATHANIEL. Office Engr., State Highway Dept., Harrisburg, Pa	Jun. Assoc. M. M.	Aug. 31, 1909 June 30, 1911 May 15, 1917
PHILLIPS, WILLIAM HALE. 508 Peyton Bldg., Spokane, Wash	Jun. Assoc. M. M.	Nov. 1, 1904 May 4, 1909 May 15, 1917
POPERT, WILLIAM HOPF. Contr. Engr., Am. Bridge Co. and U. S. Steel Products Co., Room 609, Rialto Bldg., San Francisco, Cal	Assoc. M. M.	Nov. 4, 1908 April 18, 1917
RANNEY, WILLIS. Cons. Engr. (Bartlett & Ranney), Chandler Bldg., San Antonio, Tex	Assoc. M. M.	July 9, 1912 June 12, 1917
SHAW, PERCY AUGUSTUS. 297 Grove St., Fall River, Mass	Assoc. M. M.	May 7, 1913 June 12, 1917
SHEPPERD, THOMAS SHACKELFORD. Mgr., Ulen Contr. Co., Montevideo, Uruguay	Assoc. M. M.	June 30, 1911 June 12, 1917
SPALDING, WALTER JAMES. Supt., Municipal Eng., Southern Dist. of Panama Canal, Ancon, Canal Zone, Panama	Jun. Assoc. M. M.	Nov. 1, 1904 Feb. 6, 1912 June 12, 1917
STARR, REX CAMERON. Hydr. Engr., Pacific Light & Power Corporation, 603 Pacific Elec. Bldg., Los Angeles, Cal	Assoc. M. M.	Jan. 2, 1912 May 15, 1917
TROST, ADOLPHUS GUSTAVUS. Structural Engr. (Trost & Trost), El Paso, Tex	Assoc. M. M.	Sept. 3, 1913 June 12, 1917
TRUEHART, EDWARD GARLAND. Chf. Engr., Ulen Contr. Co., Montevideo, Uruguay		Mar 13, 1917

MEMBERS (*Continued*)

		Date of Membership.
TULLOCK, HUBERT SOUTHWICK. Mgr., Highway Bridge Dept., Mo. Val. Bridge & Iron Co., Leavenworth, Kans.	Assoc. M. M.	June 4, 1913 June 12, 1917
WARNER, JACOB LATCH. Eng. Dept., E. I. du Pont de Nemours & Co., 1017 Shallcross Ave., Wilmington, Del.		May 15, 1917
WATKINS, GUY ANDERSON. Cons. Engr. (Dickinson & Watkins), 610 State Bank Bldg., Little Rock, Ark.	Jun. Assoc. M. M.	Jan. 3, 1907 Nov. 8, 1909 June 12, 1917
WHITCOMB, RALPH NIMS. Asst. to Vice-Pres., The J. G. White Eng. Corporation, 43 Exchange Pl., New York City.		May 15, 1917
WHITE, FRANK GEORGE. Chf. Engr., Board of State Harbor Commrs., Room 18, Ferry Bldg., San Francisco, Cal.	Jun. Assoc. M. M.	Dec. 3, 1901 Dec. 7, 1904 Mar. 13, 1917
WILLIAMS, MAURICE WILLIAM. Senior Asst. Engr., State Engr.'s Office, 158 State St., Albany, N. Y.	Assoc. M. M.	Jan. 31, 1911 June 12, 1917
WILLIAMS, WILLIAM HORACE. Civ. Engr. and Gen. Contr. (Doullut & Williams), 1016 Hibernia Bank Bldg., New Orleans, La.	Assoc. M. M.	Oct. 29, 1912 June 12, 1917

ASSOCIATE MEMBERS

ANDERSON, JOHN EDWARD. Lieut. and Adjutant, Royal Engrs., Headquarters, Royal Engrs., 33d Div., B. E. F., France.	Jun. Assoc. M.	Aug. 31, 1915 April 17, 1917
ATWOOD, CHESTER ELY. Res. Engr., The Valier Montana Land & Water Co., Valier, Mont.	Jun. Assoc. M.	Feb. 1, 1910 May 15, 1917
AYERS, MURRAY CHASE. 1623 West 24th St., Los Angeles, Cal.		May 15, 1917
BAKER, FREDERICK ANDREW. Chf. Engr., Bogalusa Paper Co., Inc., 308 North Border Drive, Bogalusa, La.		June 11, 1917
BANISTER, WILBUR VICK. Care, Stone & Webster, Whitinsville, Mass.		June 11, 1917
BARBER, JUSTIN FREDERIC. 1515 Mozart St., Alameda, Cal.		April 17, 1917
BECK, RALPH ERNEST. Junior Engr., Grade 8, Public Service Comm., First Dist. (Res., 14 Prospect Park, S. W.), Brooklyn, N. Y.	Jun. Assoc. M.	Dec. 3, 1913 April 17, 1917
BENNISON, ERNEST WILLIAM. County Highway and Bridge Engr., Adams County, Corning, Iowa.		Mar. 13, 1917
BERENTS, HANS. Cons. Engr., 13 Nanking Rd., Shanghai, China.		Mar. 13, 1917

ASSOCIATE MEMBERS (*Continued*)

		Date of Membership.
BICKERTON, WILBUR EARL. Designer and Estimator, Baltimore Office, Trussed Concrete Steel Co., 1123 Munsey Bldg., Baltimore, Md.	Jun. Assoc. M.	Jan. 6, 1915 May 15, 1917
BISHOP, GUY HERSEY. City Engr., Oelwein, Iowa	Jun. Assoc. M.	Dec. 2, 1914 May 15, 1917
BOGERT, JOHN RALPH. 207 Colonial Bldg., Wilkinsburg, Pa.		May 15, 1917
BOLTON, FRANK LEONARD. Res. Engr., Mill Creek Flood Control Project, 508 Palace Hardware Bldg., Erie, Pa.	Jun. Assoc. M.	June 30, 1910 May 15, 1917
BOWEN, ROBERT LAWTON. Asst. Engr., State Harbor Impvt. Comm., 26 Sycamore St., Providence, R. I.		June 11, 1917
BOWERS, ALBERT GEORGE. Gwynear, Chefoo, China		Nov. 28, 1916
BOYD, ROBERT PLATT. Res. Engr., Parish of Ouachita, Box 375, Monroe, La.		June 11, 1917
BRIGGS, ROBERT WESLEY. Asst. Engr., N. Y. C. R. R., 20 Rollins St., Yonkers, N. Y.		June 11, 1917
BROWN, JOHN HENRY, JR. Secy., Manwaring & Cummins, Inc., 24 East Church Lane, Philadelphia, Pa.		June 11, 1917
BUFORD, CHARLES HOMER. 1306 Jackson St., Sioux City, Iowa		June 11, 1917
CAMPBELL, PAUL CALDWELL. Casilla de Correo No. 403, Montevideo, Uruguay	Jun. Assoc. M.	May 6, 1914 April 17, 1917
CANFIELD, GEORGE HATHAWAY. Civ. Engr., Pacific Gas & Elec. Co., 1519 Alice St., Oakland, Cal.		May 15, 1917
CARTER, HUGH RUBEN. State Highway Engr. of Arkansas, 1869 Gaines St., Little Rock, Ark.		Mar. 13, 1917
CATON, JOHN HIRST, 3D. 70 Arnold Ave., Providence, R. I.		June 11, 1917
CHAMBERLIN, EARL WILLIAM. Engr., Fireproofing Dept., U. S. Gypsum Co., 5865 Glenwood Ave., Chicago, Ill.		May 15, 1917
CHAPIN, CHARLES WALTER. Constr. Engr., California Highway Comm., 320 Higuera St., San Luis Obispo, Cal.		June 11, 1917
CHRISTHILF, FRANCIS DORSEY. Gen. Contr. (Carozza & Christhilf), 18 York Court, Guilford, Baltimore, Md.		June 11, 1917
CLAYSON, GEORGE PHELPS. Mgr., Steel Dept., Chicago Office, Trussed Concrete Steel Co., 7455 Greenview Ave., Chicago, Ill.		June 11, 1917
CLINTON, DELMAR SMITH. Army and Navy Club, Manila, Philippine Islands		Jan. 15, 1917
CONVERSE, JOSEPH BRANDLY. Asst. State Highway Engr., State Highway Dept., Montgomery, Ala.		May 15, 1917
COURTENAY, WILLIAM HENRY. Engr., Estimator, and Designer for C. P. Bower, 1324 North 58th St., Philadelphia, Pa.		June 11, 1917

ASSOCIATE MEMBERS (*Continued*)

		Date of Membership.
CRESSON, JAMES. County Engr., Montgomery County, Airy and Church Sts., Norristown, Pa.		June 11, 1917
DE MEY, EDOUARD JEAN BERNARD. Constr. Engr., Toupet, Beil & Conley, Inc., 730 H. W. Oliver Bldg., Pittsburgh, Pa.	Jun. Assoc. M.	Jan. 2, 1912 May 15, 1917
DODGE, WALDO EDGAR. 36 Eucalyptus Rd., Berkeley, Cal.		Mar. 13, 1917
DRAPER, LOTT DAVIS. Structural Engr., Am. Bridge Co., 28 East Tulpehocken St., Germantown, Philadelphia, Pa.		May 15, 1917
DUNLAP, ARTHUR HOYT. Chf. Engr., Ward County Irrig. Dist. No. 1, Barstow, Tex.		May 15, 1917
DUNLAP, JOHN HOFFMAN. Asst. Prof. of Hydraulics and San. Eng., The State Univ. of Iowa, 104-N, Hall of Eng., Iowa City, Iowa		April 17, 1917
DUPUY, VICTOR NEWTON. Asst. Engr., Alaska Juneau Gold Min. Co., Box 186, Juneau, Alaska		Jan. 15, 1917
EASTON, RUSSELL BURNS. Cons. Engr. (Easton & Wells), Aberdeen, S. Dak.	Jun. Assoc. M.	Mar. 3, 1908 Nov. 28, 1916
EDGREN, ARTHUR H. County Engr., Lancaster County, 2045 Pepper Ave., Lincoln, Nebr.		Jan. 15, 1917
EDWARDS, RAMON SALAS. Prof., Civ. Eng., Universidad Catolica de Santiago, Compania 1618, Santiago, Chile.		April 17, 1917
ELLIS, LESTER FISHER. 52 Waltham St., Lexington, Mass.		May 15, 1917
ENGLISH, HAROLD LEWIS. Junior Structural Engr., Div. of Valuation, Interstate Commerce Comm., Washington, D. C.	Jun. Assoc. M.	Mar. 5, 1912 May 15, 1917
ERICSON, LAMBERT THEODORE. Contr. Engr., The Jennison-Wright Co., 2463 Broadway, Toledo, Ohio		May 15, 1917
EURICH, RICHARD HENDERSON. 144 Union St., Montclair, N. J.	Jun. Assoc. M.	Mar. 4, 1913 June 11, 1917
EVERETT, CHESTER MCKENZIE. (Hazen, Whipple & Fuller), 30 East 42d St., New York City		May 15, 1917
EVERS, RUDOLPH. 1138 Hancock St., Brooklyn, N. Y.		May 15, 1917
FAISON, HAYWOOD RENICK. Box 651, Wilmington, N. C.		Jan. 15, 1917
FELLOWS, PERRY AUGUSTUS. Instr. in Civ. Eng., Univ. of Michigan, 636 South 12th St., Ann Arbor, Mich.		June 11, 1917
FOX, ALVIN BARTHOLDI. (Larson & Fox), 137 Smith St., Perth Amboy, N. J.		May 15, 1917
GALE, CLARENCE STEPHENS. Res. Engr., Maryland State Roads Comm., Easton, Md.		June 11, 1917
GARNETT, BENJAMIN JAY. Office Engr., City Engr.'s Office, Spokane, Wash.	Jun. Assoc. M.	Nov. 1, 1910 Jan. 15, 1917
GIBBS, WILLIAM WETMORE. Asst. Gen. Mgr., The Phosphate Min. Co., Nichols, Fla.	Jun. Assoc. M.	Oct. 7, 1914 May 15, 1917
GIBSON, OTIS. 2407 Lincoln Way, San Francisco, Cal.		April 17, 1917

ASSOCIATE MEMBERS (*Continued*)

		Date of Membership.
GIESEN, WALTER EDWARD. Estimator, Fred A. Jones Constr. Co., 420 Interurban Bldg., Dallas, Tex		Nov. 28, 1916
GOEDER, FRANK PEAT. Asst. Prof. of Physics and Electrical Eng., State Agricultural Coll., 708 South College Ave., Fort Collins, Colo		April 17, 1917
GOODWIN, WILLIAM ROBERT. Asst. Engr., City Engr.'s Dept., 2710 Pleasant Ave., Minneapolis, Minn		May 15, 1917
GRAY, HOWARD ALLISON. Squad Chf., Stone & Webster Eng. Corporation, 71 Wallace St., West Somerville, Mass		May 15, 1917
GRUETZMACHER, CLARENCE SAYLOR. 767 Marshall St., Milwaukee, Wis		May 15, 1917
GRUNAUER, MORTIMER. Field Supt., Bing & Bing Constr. Co., 119 West 40th St. (Res., 216 West 141st St.), New York City	Jun. Assoc. M.	Feb. 6, 1912 Mar. 13, 1917
HALL, BENJAMIN MORTIMER, JR. Civ. and Min. Engr. (B. M. Hall & Son), 501 Peters Bldg., Atlanta, Ga		May 15, 1917
HANSON, NEAL. Associate and Superv. Engr., Bartlett & Ranney, Inc., Eagle's Nest Dam, Ute Park, N. Mex		June 11, 1917
HARTER, ALOYSIUS FRANK. Asst. City Engr., 726 East Culver St., Phœnix, Ariz		May 15, 1917
HARTFORD, FRED DAILEY. Chf. Draftsman, Western Chemical Mfg. Co., 608 Pearl St., Denver Colo	Jun. Assoc. M.	April 7, 1915 June 11, 1917
HERKNESS, LINDSAY COATES. Capt., Corps. of Engrs., U. S. A., Room 707, Army Bldg., New York City		Mar. 13, 1917
HEWS, WELLINGTON PRESCOTT. Asst. Field Engr., Interstate Commerce Comm., 731 Wells Fargo Bldg., San Francisco, Cal		June 11, 1917
HOFFMAN, EUGENE ROBERT. Chf. Draftsman, Washington State Highway Dept., Box 535, Olympia, Wash		May 15, 1917
HUBER, JOSEPH EARL. Div. Engr., State Highway Dept., Wise Bldg., Mt. Vernon, Ill		May 15, 1917
HULSIZER, WILLIAM HILL. Asst. Engr., Valuation Dept., U. P. R. R., Room 207, Union Pacific Headquarters, Omaha, Nebr		May 15, 1917
HUMPHREYS, EWING SLOAN. Engr., John T. Wilson Co., Mutual Bldg., Richmond, Va	Jun. Assoc. M.	Jan. 17, 1916 May 17, 1917
JOHNSON, HALBERT THEODORE. 407 Federal Bldg., Salt Lake City, Utah		April 17, 1917
JOHNSON, HOLLISTER. Junior Engr., New York State Conservation Comm., Albany, N. Y		May 15, 1917
JONES, PERCIVAL CHARLES. Res. Engr., Westinghouse, Church, Kerr & Co., 1309 Cedar Ave., Scranton, Pa.		May 15, 1917

ASSOCIATE MEMBERS (*Continued*)

Member		Date of Membership.
JONES, ROBERT LEROY. 304 California Fruit Bldg., Sacramento, Cal.		May 15, 1917
KAHLER, CHARLES PORTERFIELD. 606 Deseret News Bldg., Salt Lake City, Utah		Mar. 13, 1917
KAUFMANN, ERNST GUSTAV. Engr., Eastern Concrete Steel Co., 402 D. S. Morgan Bldg., Buffalo, N. Y.	Jun. Assoc. M.	Oct. 1, 1913 May 15, 1917
KIRSTEIN, PAUL ROBERT. Asst. Engr., City Engr.'s Office 2546 Auburn Ave., Cincinnati, Ohio		May 15, 1917
KNIGHT, ARTHUR JULIUS. Asst. Prof., Civ. Eng., and Supt., Bldg. and Grounds, Worcester Polytechnic Inst., Worcester, Mass.		June 11, 1917
KNOLLMAN, ENNO PAUL. (Braun, Fleming & Knollman), 30 East Lane Ave., Columbus, Ohio		May 15, 1917
KREAMER, ADONIS WILLIAM. U. S. Junior Engr., P. O. Box 75, Wheeling, W. Va.		May 15, 1917
LAMB, LYMAN CALVIN. 54 Quinn Ave., Youngstown, Ohio		June 11, 1917
LAMBERT, EDWARD ALLYN. Chf. Engr., New England Branch, The Austin Co., Newfield Bldg., Bridgeport, Conn.		May 15, 1917
LA ROCHE, ARTHUR LEWIS. Deputy City Engr., City Engr.'s Office, Binghamton, N. Y.		May 15, 1917
LARSON, MORGAN FOSTER. (Larson & Fox), 137 Smith St., Perth Amboy, N. J.		May 15, 1917
MACNAUGHTON, PERCIVAL JOHN. Res. Engr., Fred T. Ley & Co., Inc., 19 Warner St., Springfield, Mass.		May 15, 1917
MCCLUSKEY, WILLIAM OLIVER, JR. County Engr., Ohio County, County Bldg., Wheeling, W. Va.		May 15, 1917
MAILEY, JOHN BRUCE. 20 Howard St., East Lynn, Mass.	Jun. Assoc. M.	April 30, 1912 May 15, 1917
MANDELZWEIG, HYMAN HARRY. Asst. Engr., Cuyahoga County, 1480 East 115th St., Cleveland, Ohio		May 15, 1917
MANZANILLA Y CARBONELL, JOSÉ JUSTO. Asst. Engr., Smith, Ames & Chisholm, Lonja 510, Havana, Cuba	Jun. Assoc. M.	June 30, 1910 June 11, 1917
MELTZER, JOSEPH. 316 Dickinson St., Springfield, Mass.		April 17, 1917
MERRIAM, CHARLES ALLEN. Structural Engr. and Supt. of Constr., A. E. Doyle, 401 Worcester Bldg., Portland, Ore.		June 11, 1917
MILLS, GUY G. Acting Dist. Mgr., Portland Cement Assoc., 1123 Hurt Bldg., Atlanta, Ga.	Jun. Assoc. M.	June 24, 1914 April 17, 1917
MOLLER, IRVING CLARK. Care, The Barrett Co., 17 Battery Pl., New York City		Mar 13, 1917

ASSOCIATE MEMBERS (*Continued*)

		Date of Membership.
MURRAY, CLIFFORD EATON. Archt. and Engr., 26 Clinton St., Newark, N. J.	Jun. Assoc. M.	May 6, 1914 May 15, 1917
NELSON, ERNEST BENJAMIN. Asst. Engr., Andes Cooper Min. Co., Casilla 230, Antofogasta, Chile	Jun. Assoc. M.	Oct. 31, 1911 May 15, 1917
NICHOLSON, GEORGE FRANCES. Acting Chf. Engr., Port of Seattle, 603 Thirty-third, North, Seattle, Wash.		May 15, 1917
OLDS, ALBERT ROY. Engr., M. of W., Havana Elec. Ry., Light & Power Co., P. O. Box 570, Havana, Cuba		May 15, 1917
O'ROURKE, FRANK HUGH. Asst. Gen. Mgr., McNerney Constr. Co.; Engr. and Contr. (McNerney & O'Rourke), Sellersville, Pa.		May 15, 1917
PARKER, KINGSBURY EASTMAN. Treas. and Mgr., Clinton Constr. Co., 140 Townsend St., San Francisco, Cal.	Jun. Assoc. M.	May 5, 1908 May 15, 1917
PAXTON, THOMAS PRESTON. Commr. of Public Works, Box 937, Okmulgee, Okla.		June 11, 1917
PENN, WILLIAM CLAY. Supt., Knoxville Power Co., Alcoa, Tenn.		June 11, 1917
PERSONS, VICTOR SMITH. Sales Engr., L. A. Norris Co., 140 Townsend St., San Francisco, Cal.		June 11, 1917
PETERS, WILLIAM FREDERICK. (The Peters Carpenter Handy Co.), 206 Second National Bldg., Akron, Ohio.		April 17, 1917
PHALAN, JOHN JOSEPH FRANCIS. Engr. in Chg. of Constr. of Road 5585, State Comm. of Highways, Utica, N. Y.	Jun. Assoc. M.	April 30, 1912 May 15, 1917
PYLE, CLYDE BEETHOVEN. Asst. to Mgr., McClintic-Marshall Co., Pittsburgh, Pa.		April 17, 1917
QUINN, MAURICE JAMES. 205 Lincoln Bldg., Detroit, Mich.		May 15, 1917
RAKESTRAW, CHARLES LYSANDER. Junior Engr., U. S. Engr. Office, 405 Custom House, San Francisco, Cal.	Jun. Assoc. M.	June 4, 1913 May 15, 1917
RAMSER, CHARLES ERNEST. P. O. Box 294, Jackson, Tenn.	Jun. Assoc. M.	Mar. 1, 1910 June 11, 1917
RAY, ROBERT BRIGHT. 3017 Brighton Ave., Los Angeles, Cal.		April 17, 1917
REANEY, CHARLES FRANKLIN. Asst. Engr., Waterloo, Cedar Falls & No. Ry., 322 Denver St., Waterloo, Iowa	Jun. Assoc. M.	Aug. 31, 1915 Jan. 15, 1917
REED, PERCY LAWRENCE. Senior Instr., Civ. Eng., Drexel Inst., 268 South 38th St., West Philadelphia, Pa.		June 11, 1917
RIDDLE, WILLIAM CATHCART. Asst. Engr., Pennsylvania State Dept. of Health, Harrisburg, Pa.		June 11, 1917
RINEHART, CHARLES RAMSAY. 50 Church St., Room 2079, New York City		May 15, 1917

ASSOCIATE MEMBERS (*Continued*)		Date of Membership.
ROSCHE, WILBERT ROBIN. Office Engr., H. Koppers Co., 1109 College Ave., Pueblo, Colo		May 15, 1917
ROSS, ANDREW FRANCIS. 4167 Julian St., Denver, Colo		June 11, 1917
SAXE, VAN RENSSELAER POWELL. Pres., Standard Concrete Steel Co., Knickerbocker Bldg., Baltimore, Md	Jun. Assoc. M.	Feb. 4, 1908 Nov. 28, 1916
SCHARPENBERG, CHARLES CHRISTIAN. Efficiency Engr., Producing Dept., Standard Oil Co., 2110 Nineteenth St., Bakersfield, Cal		May 15, 1917
SCOTT, LEWIS PELOT. Asst. Engr., Illinois Highway Dept., 138 Fox St., Aurora, Ill		June 11, 1917
SECREST, THOMAS WILLIAM. Locating Engr., Alaskan Eng. Comm., Anchorage, Alaska		April 17, 1917
SHERMAN, JAMES HILTON. Vice-Pres. and Chf. Engr., Reyburn-Sherman Eng. & Constr. Co., 3600 Paseo, Kansas City, Mo		May 15, 1917
SIGLER, ELMER. Vice-Pres. and Engr., J. P. Sprague Co., 1308 Rialto Bldg., Kansas City, Mo		May 15, 1917
SILAGI, EUGENE ADALBERT. Mgr., Columbus Branch, Trussed Concrete Steel Co., 1000 Columbus Savings & Trust Bldg., Columbus, Ohio		June 11, 1917
SMITH, ELROY GEORGE. 400 Harrison Bldg., Augusta, Ga	Jun. Assoc. M.	Sept. 1, 1908 April 17, 1917
SMITH, ROY ELMER. Draftsman, Copper River & Northwestern Ry., Cordova, Alaska	Jun. Assoc. M.	June 30, 1911 April 17, 1917
SPEAR, ROY ELBERT. Engr., Sewer Design, Engr. Dept., 1316 Beach St., Flint, Mich		Mar. 13, 1917
STEELE, ISAAC CLEVELAND. Civ. Engr., Pacific Gas & Elec. Co., 445 Sutter St., San Francisco, Cal		May 15, 1917
TABER, ROCK GRANITE. Supt. of Constr., Stone & Webster, 3005 Holmes St., Dallas, Tex		June 11, 1917
THORNTON, LOUIS EARLE. Care, 1st Company, E. O. R. T. C., American Univ. Grounds, Washington, D. C		June 11, 1917
TRETTEN, OTTO CHRISTIAN. Care, California Highway Comm., Gaviota, Cal		May 15, 1917
TYSON, WILLIAM CLAUDE. Junior Engr., U. S. Engr. Office, Kansas City, Mo		May 15, 1917
VAN AUKEN, CLAUDE LINN. Asst. Engr., N. Y. C. R. R., 5256 Fulton St., Chicago, Ill		May 15, 1917
VAN VLIET, GEORGE PARKER. Chf. Engr., The Robinson Clay Product Co., 1010 East Market St., Akron, Ohio		May 15, 1917
WALKER, LESTER CARL. Asst. Chf. Engr., Idaho Irrig. Co., Ltd., Richfield, Idaho		May 15, 1917

ASSOCIATE MEMBERS (*Continued*)		Date of Membership.
WALLER, JOHN MALCOLM. Junior Structural Engr., Interstate Commerce Comm., Western Dist., Div. of Valuation, Interstate Bldg., Kansas City, Mo..........	Jun. Assoc. M.	Dec. 2, 1914 June 11, 1917
WARD, GEORGE SPARKMAN. Bowman, S. C...	Jun. Assoc. M.	April 1, 1914 Jan. 15, 1917
WARD, HERBERT KIRKMAN. Care, S. L. Ward, 2931 Dale St., San Diego, Cal..............................		May 15, 1917
WARD, JASPER DUDLEY. Co. C, 55th Infantry, Military Branch, Chattanooga, Tenn......................		Mar. 13, 1917
WARREN, MINTON MACHADO. Secy., Hydr. Div., Stone & Webster Eng. Corporation, 147 Milk St., Boston, Mass............ ..	Jun. Assoc. M.	Nov. 4, 1914 May 15, 1917
WEBSTER, MAURICE ANDERSON. With William Steele & Sons Co., 238 Winona Ave., Germantown, Philadelphia, Pa............	Jun. Assoc. M.	Sept. 2, 1914 May 15, 1917
WELLS, FRANK HARRIS. Structural Designer of Bldgs., Larrowe Constr. Co., 1199 Woodward Ave., Detroit, Mich.		April 17, 1917
WELLS, HARRY ARTEMAS. Supt., Bldgs. and Grounds, Dartmouth Coll.; Cons. Engr. and Archt., Parkhurst Administration Bldg., Hanover, N. H................		May 15, 1917
WRIGHT, CLAUDE RICHARD. Engr. Insp., U. S. Engr. Office, 802 Couch Bldg., Portland, Ore....................		April 17, 1917
YOLTON, ROBERT ELGENE. Chf. Engr., Kilby Frog & Switch Co., 2906 North 13th Ave., Birmingham, Ala........		April 17, 1917
YOUMANS, GEORGE LELAND. Supt., Puyallup Dredging Co., Box 1610, Tacoma, Wash.	Jun. Assoc. M.	Jan. 7, 1913 June 11, 1917

JUNIORS

ANDERSON, ROY LEONARD. Engr. and Salesman, California Corrugated Culvert Co., West Berkeley, Cal........	Jan. 11, 1917
ARNOLD, LEON JOHN. 1018 East 163d St., New York City..	Jan. 15, 1917
AUSTIN, HERBERT ASHFORD ROBERTSON. Junior Engr., U. S. Geological Survey, Water Resources Branch, 20 Kapiolani Bldg., Honolulu, Hawaii.............	April 17, 1917
BINGER, WALTER DAVID. Hotel Essex, Madison Ave. and 56th St., New York City......................	May 15, 1917
BRAGONIER, ARTHUR TAYLOR. 420 Campbell Ave., S. W., Roanoke, Va................................	May 15, 1917
BRICKLIN, SIMON. Draftsman, Bureau of Yards and Docks, Navy Dept., Navy Yard (Res., 62 Coming St.), Charleston, S. C..............................	May 15, 1917
BURGER, ALFRED ANDREW. Doylestown, Ohio............	June 11, 1917

JUNIORS (*Continued*)

	Date of Membership.
CARNELLI, CHARLES MICHAEL. Meter Tester, Westinghouse Elec. & Mfg. Co. of Newark, N. J., 160 East 115th St., New York City	June 11, 1917
GENUNG, JAMES HOLCOMBE, JR. 3d Co., Engrs., O. T. S., Fort Leavenworth, Kans	Jan. 15, 1917
GRAFF, GEORGE WASHINGTON. Res. Chemist and Bacteriologist, Kingston Municipal Water Supply, Box 91, R. R. 2, Kingston, N. Y	Jan. 15, 1917
GREHAN, BERNARD HENRY. 3431 Prytania St., New Orleans, La	May 15, 1917
HICKOK, CHARLIE WILLIAM. Rodman, Interstate Commerce Comm., Div. of Valuation, Interstate Bldg., Kansas City, Mo	Mar. 13, 1917
HUSSON, WILLIAM MORAGNE. 2d Lieut., 17th Cavalry, Douglas, Ariz	June 23, 1916
KNOST, WILLIAM ARNOLD. Care, United Fruit Co., Bocas del Toro, Panama	April 17, 1917
LANDRETH, JAMES TAYLOR. Asst. Engr., Bronx Parkway Comm., Scarsdale, N. Y	Nov. 28, 1916
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PIERCE, CHARLES WILLIAM. 1122 West 8th St., Los Angeles, Cal	April 17, 1917
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SELL, WILLIAM OSBORNE. Box 819, Birmingham, Ala	Jan. 15, 1917
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STEM, CLIFFORD HOEY. Asst. to City Engr., 7933 Poplar St., New Orleans, La	April 17, 1917
SWANSON, WILLIAM ROBERT. Engr., Flat Slab Eng. Co., 1049 Otis Bldg., Chicago, Ill	June 11, 1917
WALKER, WATSON FRANK. San. Engr., Detroit Board of Health, Detroit, Mich	Jan. 15, 1917
WEBER, WALTER RAYMOND. 52 South Lincoln, Denver, Colo.	May 15, 1917

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WRENN, JAMES FRANCIS. Pres. and Gen. Mgr., McGuire Constr. Co., Inc., Box 247, Norfolk, Va.

JUNIORS

ASHKINS, NATHAN THOMAS. Gen. Mgr., Jefferson Cement Tile Co., Weymouth, N. S., Canada.

ATKINSON, GUY. Care, The Am. Rolling Mill Co., Middletown, Ohio.

BANBROOK, HERRMANN. Structural Engr., Ford Motor Co., 226 Marston Ave., Detroit, Mich.

BETTS, CLIFFORD AULL. 4 West 43d St., St. Elmo, Tenn.

BLEY, CHARLES NICHOLAS. Raymond, Wash.

BOLIN, HARRY WILLIAM. Structural Draftsman, Roberts & Schaefer, McCormick Bldg. (Res., 3905 Sheridan Rd.), Chicago, Ill.

BROWN, HORATIO WHITTEMORE. Elm St., Concord, Mass.

CARPENTER, SINCLAIR ERNEST. 515 Forest St., Oakland, Cal.

CLAUSEN, STANLEY JAMES. With Cameron, Joyce & Co., 17 South 7th St., Keokuk, Iowa.

COBB, WILLIAM RICHARD. 2973 Folsom St., San Francisco, Cal.

CONNOLLY, DONALD HILARY. Capt., Corps of Engrs., U. S. A., 1st Battalion, Mounted Engrs., El Paso, Tex.

COOK, HOLTON. 304 Commonwealth Bldg., Jackson, Mich.

COOMBS, DONALD GLADSTONE. Asst. to Supt. of Constr., U. S. Reclamation Service, Rimrock, Wash.

DALEE, WILLIAM AMON. Care, Canadian Des Moines Steel Co., Ltd., Chatham, Ont., Canada.

DE CHARMS, RICHARD, JR. 3d Co., Engr. Officers' Training Camp, American Univ., Washington, D. C.

DEISER, NORMAN ARTHUR. Supt., John B. Roberts, 86th St. and Lexington Ave., New York City (Res., 382 New York Ave., Brooklyn, N. Y.).

DENHAM, DONALD POWER. 397 Reid St., Peterboro, Ont., Canada.

DOUGHERTY, EDWARD JAMES. Supt., Empire Eng. Co., Inc., Fallsington, Pa.

DOW, HEZEKIAH SHAILER. Asst. Engr., Rap. Trans. Subway Constr. Co., 190 Archer Ave., Mount Vernon, N. Y.

DREYFUS, SAMUEL CELLNER. Highway Engr., 1208 Empire Bldg., Atlanta, Ga.

DRURY, WALTER RHODES. 426 Maynard St., Ann Arbor, Mich.

JUNIORS (*Continued*)

ESTES, LEWIS ALDEN. Dist. Representative, Trussed Concrete Steel Co., 146 Summer St., Boston, Mass.

FINDLAY, ELWIN HAROLD. 175 West 73d St., New York City.

FISCHER, CHARLES, JR. 7th Co., 2d N. Y. Regiment, Plattsburg, N. Y.

GAY, GEORGE INNESS. Care, The Comm. for Relief in Belgium, 3 London Wall Bldgs., London, E. C., 2, England.

GUNTHER, HERMAN DIETRICH. Care, J. F. Shanley Co., P. O. Box 21, Horseheads, N. Y.

HABERLE, EDWARD LOUIS. 1310 Emerson Ave., North, Minneapolis, Minn.

HARRINGTON, WELLESLEY CARL. Weedsport, N. Y.

HAYS, JAMES BUCHANAN. Designing Draftsman with F. C. Horn, Meridian, Idaho.

HESLOP, PAUL LOVERIDGE. Fargo Eng. Co., Jackson, Mich.

HEWETT, MAURICE WILLIAM. First Engr. Company, 13th Provisional Regiment, Fort Leavenworth, Kans.

HOWARD, CECIL WARD. 210 Allendale Ave., Detroit, Mich.

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LARIMER, ROBERT SHERMAN. 1214 Maple Ave., Evanston, Ill.

LEHRBACH, HENRY GARDNER. 363 Dearborn St., Buffalo, N. Y.

LEONARD, EDWARD PHILIP. 401 Macon St., Brooklyn, N. Y.

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MCGEE, HAROLD GILBERT. Care, Lucas County San. Eng. Dept., Toledo, Ohio.

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MATTHEW, RAYMOND. Asst. Engr., Idaho Irrig. Co., Richfield, Idaho.

MILHAN, DAVID NELSON. Div. Engr., Portland Cement Assoc., Sodus, N. Y.

MILKOWSKI, VICTOR JOHN. 2d Lieut., Engr. Officers' Reserve Corps, Engr. Co. No. 1, Madison Barracks, N. Y.

MUNSON, CHARLES HAROLD. Care, W. H. Munson, 317 Franklin St., Winona, Minn.

NAGLER, FLOYD AUGUST. 387 Hamilton St., Albany, N. Y.

NEWKIRK, SAMUEL FRANK. Lamartine Ave., Bayside, N. Y.

OSTROM, CHARLES DOUGLAS YELVERTON. 2d Lieut., Coast Artillery Corps, U. S. A., Fort Barrancas, Fla.

PARSONS, MAURICE GIESY. Purchasing Agt., Peninsula Shipbuilding Co., Portland, Ore.

PEARCE, RUFUS BURLESON. Richland, Tex.

RICE, ROGER CUSHING. Care, U. S. Geological Survey, 25 Federal Bldg., Topeka, Kans.

RICHARDS, GEORGE WILLIAM. 531 Pennsylvania Ave., Oakmont, Pa.

JUNIORS (*Continued*)

SCHROEDER, SEATON, JR. Lieut., U. S. N. R. F., Public Works Dept., Navy Yard, Brooklyn, N. Y.

SEARIGHT, GEORGE PETER. With The Foundation Co., Ltd., 127 Drummond St., Montreal, Que., Canada (Res., Carlisle, Pa.).

SMITH, RICHARD BENNETT. 1451 Stout St., Denver, Colo.

STANLEY, WILLIAM EDWARD. Care, J. W. Stanley, Burrton, Kans.

STEINBRUCH, WALTER. Care, Snare & Triest Co., Chattanooga, Tenn.

STEPHENS, UEL. Box 133, Lometa, Tex.

SUMMERS, RICHARD ELVIN JEWELL. Civ., Hydr. and San. Engr., 508 McNair Ave., Wilkinsburg, Pa.

SWARTZ, LEON. Master Engr., Engr. Reserve, Broad Ford, Pa.

TAYLOR, SENECA VERN. Dist. Sales Mgr., The General Fireproofing Co., 73 State St., Room 205, Detroit, Mich.

TILTON, GEORGE ALBERT, JR. Care, P. L. & P. Co., Big Creek, Fresno, Cal.

TOMS, JAY WILLIAM. State Sanatorium, Md.

VAN NESS, RUSSELL ALGER. McLean, Ill.

VELTFORT, THEODORE ERNST. Care, H. E. Allen, 131 King St., Dorchester, Mass.

WAY, WILLIAM FLOYD. Sergeant-Draftsman, Co. D, 8th Regiment, Reserve Engrs., 4730 Eleventh Ave., N. E., Seattle, Wash.

WHITE, DAVID EWING. 4846 Magnolia Ave., Apartment No. 2, Chicago, Ill.

WHITNEY, RALPH EDWARD. 154 Washington St., Keene, N. H.

WONG, JICK GAM. Hankow-Ichang Ry., Res. Engr.'s Office, Section 10, Yang-Kia-Hung, Hupeh Province, China.

WOOLWORTH, WENDELL HOWARD. 2d Lieut., U. S. A., Commanding Co. K, 28th Infantry, Mission, Tex.

YEO, WILLIAM ALBERT. Civ. Engr., Nicaragua Development Corporation, Pearl Lagoon, Nicaragua.

YEREANCE, ALEXANDER WOODWARD. 418 Centre St., South Orange, N. J.

REINSTATEMENTS

MEMBERS	Date of Reinstatement.
ROSS, ALEXANDER BELL	June 11, 1917

ASSOCIATE MEMBERS

GOODSELL, DANIEL BERTHOLF	June 11, 1917
TURLEY, OMNER JAY	June 11, 1917

JUNIORS

HORRIGAN, WILLIAM JAMES	June 11, 1917

RESIGNATIONS

ASSOCIATE MEMBERS	Date of Resignation.
MOOREFIELD, CHARLES HENRY	June 11, 1917
JUNIORS	
SOUTHER, MORTON EDWIN	June 11, 1917

DEATHS

EVEREST, CHARLES MARVIN. Elected Fellow, November 1st, 1892; died July 22d, 1917.

FIRMSTONE, FRANK. Elected Member, August 7th, 1878; died June 27th, 1917.

FRAZEE, JOHN HATFIELD. Elected Associate Member, December 6th, 1899; died May 4th, 1917.

GRIMSHAW, JAMES WALTER. Elected Member, November 7th, 1888; died February 15th, 1917.

HOWARD, JOEL MANNING. Elected Associate Member, June 4th, 1913; died May 22d, 1917.

JENKINS, JAMES EDGAR. Elected Associate Member, December 5th, 1906; Member, March 14th, 1916; died July 5th, 1917.

LOCKE, FRANKLIN BUCHANAN. Elected Member, March 1st, 1893; died May 11th, 1917.

MILLER, STANLEY ALFRED. Elected Junior, February 4th, 1902; Associate Member, April 6th, 1909; Member, June 24th, 1916; died May 13th, 1917.

POMEROY, LEWIS ROBERTS. Elected Associate, April 2d, 1890; died May 7th, 1917.

SIMSON, DAVID. Elected Member, January 8th, 1902; died December 16th, 1916.

SPENCE, DAVID WENDEL. Elected Member, October 1st, 1913; died June 29th, 1917.

Total Membership of the Society, August 2d, 1917, 8 401.

MONTHLY LIST OF RECENT ENGINEERING ARTICLES OF INTEREST

(May 1st to June 30th, 1917)

NOTE.—*This list is published for the purpose of placing before the members of this Society, the titles of current engineering articles, which can be referred to in any available engineering library, or can be procured by addressing the publication directly, the address and price being given wherever possible.*

LIST OF PUBLICATIONS

In the subjoined list of articles, references are given by the number prefixed to each journal in this list:

(2) *Proceedings*, Engrs. Club of Phila., Philadelphia, Pa.
(3) *Journal*, Franklin Inst., Philadelphia, Pa., 50c.
(4) *Journal*, Western Soc. of Engrs., Chicago, Ill., 50c.
(5) *Transactions*, Can. Soc. C. E., Montreal, Que., Canada.
(7) *Gesundheits Ingenieur*, München, Germany.
(8) *Stevens Indicator*, Hoboken, N. J., 50c.
(9) *Industrial Management*, New York City, 25c.
(11) *Engineering* (London), W. H. Wiley, 432 Fourth Ave., New York City, 25c.
(12) *The Engineer* (London), International News Co., New York City, 35c.
(13) *Engineering News-Record*, New York City, 15c.
(15) *Railway Age Gazette*, New York City, 15c.
(16) *Engineering and Mining Journal*, New York City, 15c.
(17) *Electric Railway Journal*, New York City, 10c.
(18) *Railway Review*, Chicago, Ill., 15c.
(19) *Scientific American Supplement*, New York City, 10c.
(20) *Iron Age*, New York City, 20c.
(21) *Railway Engineer*, London, England, 1s. 2d.
(22) *Iron and Coal Trades Review*, London, England, 6d.
(23) *Railway Gazette*, London, England, 6d.
(24) *American Gas Engineering Journal*, New York City, 10c.
(25) *Railway Mechanical Engineer*, New York City, 20c.
(26) *Electrical Review*, London, England, 4d.
(27) *Electrical World*, New York City, 10c.
(28) *Journal*, New England Water-Works Assoc., Boston, Mass., $1.
(29) *Journal*, Royal Society of Arts, London, England, 6d.
(30) *Annales des Travaux Publics de Belgique*, Brussels, Belgium, 4 fr.
(31) *Annales* de l'Assoc. des Ing. Sortis des Ecoles Spéciales de Gand, Brussels, Belgium, 4 fr.
(32) *Mémoires et Compte Rendu des Travaux*, Soc. Ing. Civ. de France, Paris, France.
(33) *Le Génie Civil*, Paris, France, 1 fr.
(34) *Portefeuille Economiques des Machines*, Paris, France.
(35) *Nouvelles Annales de la Construction*, Paris, France.
(36) *Cornell Civil Engineer*, Ithaca, N. Y.
(37) *Revue de Mécanique*, Paris, France.
(38) *Revue Générale des Chemins de Fer et des Tramways*, Paris, France.
(39) *Technisches Gemeindeblatt*, Berlin, Germany, 0, 70m.
(40) *Zentralblatt der Bauverwaltung*, Berlin, Germany, 60 pfg.
(41) *Electrotechnische Zeitschrift*, Berlin, Germany.
(42) *Proceedings*, Am. Inst. Elec. Engrs., New York City, $1.
(43) *Annales des Ponts et Chaussées*, Paris, France.
(44) *Journal*, Military Service Institution, Governors Island, New York Harbor, 50c.
(45) *Coal Age*, New York City, 10c.
(46) *Scientific American*, New York City, 15c.
(47) *Mechanical Engineer*, Manchester, England, 3d.
(48) *Zeitschrift*, Verein Deutscher Ingenieure, Berlin, Germany, 1, 60m.
(49) *Zeitschrift für Bauwesen*, Berlin, Germany.
(50) *Stahl und Eisen*, Düsseldorf, Germany.
(51) *Deutsche Bauzeitung*, Berlin, Germany.
(52) *Rigasche Industrie-Zeitung*, Riga, Russia, 25 kop.
(53) *Zeitschrift*, Oesterreichischer Ingenieur und Architekten Vereines, Vienna, Austria, 70h.
(54) *Transactions*, Am. Soc. C. E., New York City, $12.
(55) *Transactions*, Am. Soc. M. E., New York City, $10.
(56) *Transactions*, Am. Inst. Min. Engrs., New York City, $6.
(57) *Colliery Guardian*, London, England, 5d.
(58) *Proceedings*, Engrs.' Soc. W. Pa., 568 Union Arcade Bldg., Pittsburgh, Pa., 50c.

(59) *Proceedings*, American Water-Works Assoc., Troy, N. Y.
(60) *Municipal Engineering*, Indianapolis, Ind., 25c.
(61) *Proceedings*, Western Railway Club, 225 Dearborn St., Chicago, Ill., 25c.
(62) *American Drop Forger*, Thaw Bldg., Pittsburgh, Pa., 10c.
(63) *Minutes of Proceedings*, Inst. C. E., London, England.
(64) *Power*, New York City, 5c.
(65) *Official Proceedings*, New York Railroad Club, Brooklyn, N. Y., 15c.
(66) *Gas Journal*, London, England, 6d.
(67) *Cement and Engineering News*, Chicago, Ill., 25c.
(69) *Der Eisenbau*, Leipzig, Germany.
(71) *Journal*, Iron and Steel Inst., London, England.
(71*a*) *Carnegie Scholarship Memoirs*, Iron and Steel Inst., London, England.
(72) *American Machinist*, New York City, 15c.
(73) *Electrician*, London, England, 18c.
(74) *Transactions*, Inst. of Min. and Metal., London, England.
(75) *Proceedings*, Inst. of Mech. Engrs., London, England.
(76) *Brick*, Chicago, Ill., 20c.
(77) *Journal*, Inst. Elec. Engrs., London, England, 5s.
(78) *Beton und Eisen*, Vienna, Austria, 1, 50m.
(79) *Forscherarbeiten*, Vienna, Austria.
(80) *Tonindustrie Zeitung*, Berlin, Germany.
(81) *Zeitschrift für Architektur und Ingenieurwesen*, Wiesbaden, Germany.
(83) *Gas Age*, New York City, 15c.
(84) *Le Ciment*, Paris, France.
(85) *Proceedings*, Am. Ry. Eng. Assoc., Chicago, Ill.
(86) *Engineering-Contracting*, Chicago, Ill., 10c.
(87) *Railway Engineering and Maintenance of Way*, Chicago, Ill., 10c.
(88) *Bulletin* of the International Ry. Congress Assoc., Brussels, Belgium.
(89) *Proceedings*, Am. Soc. for Testing Materials, Philadelphia, Pa., $5.
(90) *Transactions*, Inst. of Naval Archts., London, England.
(91) *Transactions*, Soc. Naval Archts. and Marine Engrs., New York City.
(92) *Bulletin*, Soc. d'Encouragement pour l'Industrie Nationale, Paris, France.
(93) *Revue de Métallurgie*, Paris, France, 4 fr. 50.
(95) *International Marine Engineering*, New York City, 20c.
(96) *Canadian Engineer*, Toronto, Ont., Canada, 10c.
(98) *Journal*, Engrs. Soc. Pa., Harrisburg, Pa., 30c.
(99) *Proceedings*, Am. Soc. of Municipal Improvements, New York City, $2.
(100) *Professional Memoirs*, Corps of Engrs., U. S. A., Washington, D. C., 50c.
(101) *Metal Worker*, New York City, 10c.
(102) *Organ für die Fortschritte des Eisenbahnwesens*, Wiesbaden, Germany.
(103) *Mining and Scientific Press*, San Francisco, Cal., 10c.
(104) *The Surveyor and Municipal and County Engineer*, London, England, 6d.
(105) *Metallurgical and Chemical Engineering*, New York City, 25c.
(106) *Transactions*, Inst. of Min. Engrs., London, England, 6s.
(107) *Schweizerische Bauzeitung*, Zürich, Switzerland.
(108) *Southern Machinery*, Atlanta, Ga. 10c.
(109) *Journal*, Boston Soc. C. E., Boston, Mass., 50c.
(110) *Journal*, Am. Concrete Inst., Philadelphia, Pa., 50c.
(111) *Journal of Electricity, Power and Gas*, San Francisco, Cal., 25c.
(112) *Internationale Zeitschrift für Wasser-Versorgung*, Leipzig, Germany.
(113) *Proceedings*, Am. Wood Preservers' Assoc., Baltimore, Md.
(114) *Journal*, Institution of Municipal and County Engineers, London, England, 1s. 6d.
(115) *Journal*, Engrs.' Club of St. Louis, St. Louis, Mo., 35c.
(116) *Blast Furnace and Steel Plant*, Pittsburgh, Pa., 15c.

LIST OF ARTICLES

Bridges.

The Reconstruction of Koari Bridge, Great Indian Peninsula Railway. Thomas Christie Hood. (63) 1915-16, Pt. II.
Pier-Foundations of the Wanganui Bridge, New Zealand.* Robert West Holmes. (63) 1915-16, Pt. II.
Design of Flat Arches with Fixed Ends. G. R. Maguel. (11) Apr. 27.
Graphics in Bridge Design.* H. S. Jacoby. (36) May.
Salvaging Second-Hand Girder Spans.* S. J. Corey. (87) May.
Modern Heavy Traffic Destroys East River Bridge Paving.* (13) May 10.
Will Soon Complete Sciotoville Continuous Truss Bridge.* (13) May 17.
Concrete Arch Bridge for Railway Crosses River on Reverse Curve.* (13) May 24.
Suspension Bridge Solves Problem of Crossing Rio Chiriqui in Panama.* A. S. Zinn. (13) May 31.

* Illustrated.

Bridges—(Continued).

The New Burlington Bridge at Kansas City.* (15) June 8.
Tendencies in Bridge Construction: An Interview.* (13) June 14.
Bridge Renewals Under Abnormal Conditions.* (15) June 22.
Restauration de Ponts et Béton Armé Fissuré sur les Chemins de Fer prussiens.* (33) June 2.
Wettbewerb für eine Brücke über die Birs an der Redingstrasse in Basel.* (107) May 5.

Electrical.

The Mechanical Working of Lead-Covered Underground Telephone-Cables. Charles Frederick Street. (63) 1915-16, Pt. II.
Electrical Precipitation of Solids from Gases. Linn Bradley. (5) Jan.-June, 1916.
Star-Delta Method of Starting Three-Phase Motors.* (111) Serial beginning Feb. 1.
The Determination of the Sequence of Phases from Wattmeter Readings.* Gisbert Kapp. (77) Apr.
Machine-Switching Telephone Gear.* F. R. McBerty. (77) Apr.
New Apparatus and Appliances.* (27) Apr. 5.
Impedance of Steel Rails.* (27) Apr. 5.
Conductivity of Earth Conductors and Metallic Sheathing. S. Simon. (Paper read before Assoc. of Min. Elec. Engrs.) (22) Apr. 13.
Wheatstone Bridges and Some Accessory Apparatus for Resistance Thermometry.* E. F. Mueller. (Abstract of paper in *Bulletin* of Bureau of Standards.) (73) Apr. 13.
The Lumen as a Measure of Illuminating Power.* (26) Apr. 13.
Large Generator and Transformer Failures on the Rand Power Companies Systems.* A. E. Val Davies. (73) Apr. 13.
The Greaves-Etchells Electric Furnace.* (26) Apr. 13.
Tramways with Rectified Current.* (Abstract of article in the Brown-Boveri *Mitteilungen.*) (73) Apr. 13.
A Linking-up Scheme: Accrington, Haslingden and Rawtenstall.* (26) Apr. 20.
New Electrical Devices, Fittings and Plant.* (26) Apr. 20.
The Electrical Equipment of Motor Garages. (26) Apr. 20.
Range of Wireless Stations. R. Chenevix Trench. (73) Serial beginning Apr. 20.
An Electric Resistance Furnace for Melting in Crucibles.* H. C. Greenwood and R. S. Hutton. (Paper presented before the Inst. of Metals.) (47) Apr. 20.
Electric Vehicles.* C. G. Conradi. (Abstract of paper read before Derby Soc. of Engrs.) (73) Apr. 27.
High-Tension Overhead Transmission Lines. G. V. Twiss. (Abstract of paper read before Inst. of Elec. Engrs.) (73) Apr. 27; (26) Serial beginning May 11; (77) June.
Voltage Regulation of Rotary Converters.* G. A. Juhlin. (Paper read before the Inst. of Elec. Engrs.) (26) Apr. 27; (77) Apr.
Wiring with Tough Rubber Compound Cables.* W. Ellerd-Styles. (26) Apr. 27.
Electric Equipment of a 35-In. Reversing Mill. (22) Apr. 27.
Novel Features in D. C. Watt-Hour-Meter Construction. (26) Apr. 27.
Wayleaves. C. Vernier. (Paper and discussion read before Inst. of Elec. Engrs.) (77) Serial beginning May; (104) Serial beginning Apr. 20; (26) Serial beginning Apr. 27; (73) Serial beginning Apr. 27.
A New Concentric Standard Dynamometer Wattmeter for Heavy Currents: and Concentric Non-inductive Standards of Low Resistance.* A. E. Moore. (77) May.
Some Points in Connection with Engineering Specifications. J. Shepherd. (77) May.
The Insulator Situation.* W. D. Peaslee. (42) May.
Multiplexing in Cable Telegraphy.* George O. Squier. (3) May.
Electric Furnace in Making Special Steel.* Charles C. Lynde. (116) May.
Expansion Effects as a Cause of Deterioration in Suspension Type Insulators.* J. A. Brundige. (42) May.
Unusual Details in Power House Installation.* J P. Jollyman. (111) May 1.
Mechanically Operated Electric Elevator Controller.* J. Gintz, Jr. (64) May 1.
Equipment Necessary for the Cottrell Process.* (111) May 1.
Promotion in Electricity Supply Undertakings. (26) May 4.
Note on the Rating of Intermittent Motors of Large Output.* H. H. Broughton. (73) May 4.
February Central Station Statistics. (27) May 5.
Practical Talks on Controllers-Series-Type Contactors.* (64) May 8.
The Wiring Rules of the Institution of Electrical Engineers. W. R. Rawlings. (Abstract of paper read before Assoc. of Supervising Electricians.) (47) May 11; (26) Apr. 27.
The Determination of the Torque of a Direct-Current Meter.* E. Alberti. (Abstract from *Elektrotechnische Zeitschrift.*) (73) May 11.
Electro-Culture and Crop Production: Results During 1916.* (26) May 11.

* Illustrated.

Electrical—(**Continued**).

Electric Converting Machinery.* C. S. Buyers. (22) May 11.
Overhead Lines and Inductive Interference. (Abstract of report submitted to Nat. Elec. Light Assoc.) (27) May 12.
Connecticut Company's Power Plant.* (17) May 12.
Floodlighting Outdoor Substations.* M. M. Samuels. (27) May 12.
The Equilibrium of the Basic Bismuth Salts. A. Mutscheller. (105) May 15.
The Production of Magnetite Electrodes. M. De Kay Thompson and T. C. Atchison. (105) May 15.
Demand Meters of Time Lag Type.* W. A. Hillebrand. (111) May 15.
Hyperbolic Solution for Engineering Problems.* W. D. Peaslee. (111) Serial beginning May 15.
Steam *vs.* Oil Engines for Small Light Plant.* L. H. Morrison. (64) May 15.
The Ossman Phenomenon. William C. Moore. (Paper read before the Am. Electrochemical Soc.) (105) May 15.
Emergency Transformer Connections.* G. P. Roux. (64) May 15.
Principles of Power-Plant Management.* Walter N. Polakov. (Abstract of paper read before joint meeting of Boston Sections of Am. Soc. of Mech. Engrs. and Am. Inst. of Elec. Engrs.) (64) May 15.
Practical Talks on Controllers—Lockout-Type Contactors.* (64) Serial beginning May 15.
15 000-K. W. Three-Phase Turbo-Alternator For Lots-Road Power Station.* (11) Serial beginning May 18.
Experiments on Construction of Electric Bells. L. H. Howlett and W. E. King. (22) May 18.
Speed Control of Induction Motors for Steel Mill Drive. J. D. Wright. (Abstract from *General Electric Review.*) (73) May 18.
The Theory of Armature Windings.* S. P. Smith. (Abstract from *Proceedings*, Inst. of Elec. Engrs.) (73) Serial beginning May 18.
X-Rays. William J. Hancock. (Paper read before Royal Soc. of Western Australia.) (19) May 19.
Three-Phase Connections.* R. E. Uptegraff. (27) May 19.
Some Causes of Underground Cable Failure.* Henry A. Cozzens, Jr. (27) May 19.
Reconnecting Induction Motors—Diagrams of Uncommon Open-Slot Windings.* A. M. Dudley. (64) May 22.
The Electrical Study Course—Series- and Parallel-Circuit Calculations.* (64) Serial beginning May 22.
Providing for Telephone Service Requirements in Design of Large Buildings. (86) May 23.
Electric Japanning. C. D. Carlson. (Abstract of paper presented before Cleveland Eng. Soc.) (20) May 24.
Furnace Charging Machine.* (20) May 24.
Surge Breakdowns in Networks. M. J. Sarolea. (Abstract from *Revue Générale de l'Electricité.*) (26) May 25; (27) June 30.
Characteristics of Maximum-Demand Meters. W. F. Kimball. (27) May 26.
How to Detect and to Deal with Electricity Thieves.* (27) May 26.
Costliness of Light Absorption.* Roy Kegerreis. (27) May 26.
Tentative Draft of a Code of Safety Standards for Power-Transmission Machinery.* (55) June.
Electric Wave Phenomena in the Dynamo-Electric Machine.* F. Creedy. (77) June.
Electric Motor Control Gear. John T. Mould. (Abstract of paper read before Assoc. of Supervising Electricians.) (47) June 1.
The Protection of Telephone Circuits Used in Electric Power Distribution.* E. K. Shelton. (Abstract from *General Electric Review.*) (73) June 1.
A Noteworthy Symposium on Rates. (Discussion held at Pacific Coast Section of N. E. L. A.) (111) June 1.
A Self-Recording Electrometer For Atmospheric Electricity.* W. A. Douglas Rudge. (Abstract of paper in *Proceedings*, Cambridge Philosophical Soc.) (73) June 1.
Definite Time Interval Demand Meters.* W. A. Hillebrand. (111) June 1.
The Standardisation of Resistors.* (73) June 1.
Static Sub-Stations. (26) June 1.
Electric Cooking and Water Heating. (Discussion held at Pacific Coast Section of N. E. L. A.) (111) June 1.
The Manufacture of Controller Drums.* R. K. Shield. (26) June 1.
The Resistance of Earth Connections.* L. Birks and E. Webb. (Abstract of paper read before Philosophical Inst. of Canterbury, New Zealand.) (73) June 1.
Some Theoretical Aspects of Electrical Fume Precipitation.* W. W. Strong. (Paper presented at joint meeting of Am. Inst. of Min. Engrs. and Am. Electro-Chemical Soc.) (105) June 1.
What Adequate Lighting Means to Industries.* C. E. Clewell. (27) June 2.

* Illustrated.

Electrical—(Continued).

Chart for Determining Power Costs.* H. Bailey. (64) June 5.
Control of Direct-Current Shunt Motors.* H. D. James. (Abstract of paper read before Am. Inst. of Elec. Engrs.) (47) June 8.
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Reinforced Concrete Transmission Poles.* (26) June 8.
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Economical Spacing of Transformers. P. O. Reyneau. (27) June 16.
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Military and Technical Considerations of Battleship Design. R. D. Gatewood. (91) Vol. 24, 1916.
On the Suitability of Current Design of Submarines to the Needs of the United States Navy.* N. L. Rogers. (91) Vol. 24, 1916.
Salvage Equipment Used in Raising Submarine *F-4.** J. A. Furer. (91) Vol. 24, 1916.
The Rolling and Floating Steel Cassions of the Levis Dry Dock, Lauzon, County of Levis, P. Q.* Lesslie R. Thomson. (5) Jan.-June, 1916.

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By-Product Coke and Coking Operations.* C. J. Ramsburg and F. W. Sperr, Jr. **(55)** June.

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Leather—Treatment and Manufacture. Lloyd Balderston. (Lecture delivered before Phil. Section of Am. Chemical Soc.) **(2)** June.

Selection of Machine Tools. Willard Doud. **(25)** June.

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Electric Welding on the Rock Island.* E. Wanamaker. **(25)** June.

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Effusion Method of Determining Gas Density.* Junius David Edwards. **(83)** June 1.

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Country Drawing Upon Stored Petroleum Reserve. V. H. Manning. (From Address before Editorial Conference of the Business Publishers' Assoc.) **(24)** June 9.

Extraction of Light Oils on a Small Scale.* Norton H. Humphrys. **(24)** June 9.

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Economizer Explosion Explained.* **(64)** June 12.

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The Steam Calorimeter and Its Use in Fuel Oil Practice.* Robert Sibley and Chas. H. Delany. **(111)** June 15.

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Producer Gas and Its Industrial Uses. F. W. Steere. (Paper read before Soc. of Detroit Chemists, Am. Chem. Soc.) (105) June 15; (20) June 7.
The After-War Development of the Gas Industry.* Alph. Mailhe. (Translated by Frederick W. Scholz from *Revue Generale des Sciences.*) (83) Serial beginning June 15.
Gas-Fired Boilers. William B. Woodhouse. (73) June 15.
Anthracite Coal. W. H. Booth. (26) Serial beginning June 15.
Production and Consumption of Gasoline in the United States. Van. H. Manning. (Address before Editorial Conference of the Business Publishers Assoc.) (83) June 15.
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Baltimore Company's Appliance Testing Laboratory Protects Interests of Consumer and, at the Same Time, is of Benefit to the Manufacturer.* H. M. Riley. (24) June 16.
The Foundry Today. Benj. D. Fuller. (Paper presented before Cleveland Eng. Soc.) (19) June 16.
New Woods for Paper Pulp.* Otto Kress. (46) June 16.
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Future Lines of Advance in Coking Practice. George E. Foxwell. (Abstract of Paper read before Yorkshire Section of Soc. of Chemical Industry.) (66) June 19.
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The Spontaneous Firing of Coal. J. S. Haldane. (Abstract of paper read before Inst. of Min. Engrs.) (22) June 22; (57) June 22.
Fuel Economy. J. A. Robertson. (Abstract of Paper read before Convention of Incorporated Municipal Elec. Assoc.) (73) June 22; (57) June 29.
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Experiences in the Distribution of Mixed Gases at Los Angeles, Cal. Norton H. Humphrys. (66) June 26.
A Discussion on Fuel Economy. (Discussion of J. A. Robertson's Paper read before Incorporated Municipal Elec. Assoc.) (66) June 26.
Continuous Removal of Soot.* (64) June 26.
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Welds—The Weakness and Merits of Their Structure.* P. A. E. Armstrong. (64) June 26.
Defects in Finished Rolled Steel.* George W. Dress. (20) June 28.
Equivalent Cost of Coal and Oil as Fuel.* R. L. Wales. (16) June 30.
An Analysis of the Tidewater Pooling Plan. (45) June 30.
Instructions Pratique pour la Détermination du Pouvoir Calorifique du Gaz.* Ville de Paris, Services Généraux d'Eclairage. (43) Mar.
La Culture Mécanique en France.* Ch. Dantin. (33) April 14.
Les Electro-Aimants de Levage.* J. Vichniak. (33) April 21.
Les Installations pour le Transbordement et l'Emmagasinage du Charbon aux Extrémités du Canal de Panama. A. Dumas. (33) June 9.
Nouvelles Fonderies Américaines, Fonderies de Kenosha (Wisconsin) et de Cranston (New England).* (33) June 30.
Dampferzeugung durch Elektrizität mit Wärme-Aufspeicherung.* E. Höhn. (107) Apr. 28.
Bericht über neue Geschwindigkeits-Regulatoren, Modell 1916, von Escher Wyss & Cie., Zurich.* Franz Prásil. (107) Serial beginning June 9.

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Potash as One Blast Furnace By-Product. R. J. Wysor. (116) April.
Silica as a Component of Furnace Slags.* Wallace G. Imhoff. (116) April.
Roll Scale as a Factor in Bessemerizing. A. Patton and F. N. Speller. (116) April.
Refractory Linings and Materials. J. W. Haulman. (116) Serial beginning April.
Coal-Gas as a Fuel for Melting Non-Ferrous Alloys.* George Bernard Brook. (Abstract of paper read before the Inst. of Metals.) (47) Apr. 20.
An Electric Resistance Furnace for Melting in Crucibles.* H. C. Greenwood and R. S. Hutton. (Paper presented before the Inst. of Metals.) (47) Apr. 20.
The Use and Abuse of Steel.* R. K. Bagnall-Wild and E. W. Birch. (12) Serial beginning April 27; (47) Serial beginning May 25.
The German Steel Syndicate and the British Steel Industry. W. J. Ashley. (Abstract of Paper read before Staffordshire Iron and Steel Inst.) (22) April 27.
Ferromanganese in the Iron and Steel Industry.* Robert J. Anderson. (3) May.
Requirements in the Treating of Materials. F. F. Beall. (62) May.
Characteristics of Electric Furnace Slags. Wallace G. Imhoff. (116) May.
The Penetration of the Hardening Effect in Chromium and Copper Steels. L. Grenet. (Abstract of Paper read before Iron and Steel Inst.) (22) May 4.
The Influence of Surface Tension Upon the Properties of Metals, Especially of Iron and Steel. F. C. Thompson. (Abstract of Paper read before Iron and Steel Inst.) (22) May 4.
Percentage of Oil in Flotation. Herbert A. Megraw. (16) May 5.
Counter-Current Decantation System.* Algernon Del Mar. (16) May 5.
Machinery and Steel Plant Labor. Charles Reitell. (20) May 10.
Modern Concentration of Colorado Tungsten Ores.* S. Fischer, Jr. (105) Serial beginning May 15.
The Electrolytic Pickling of Steel. M. De Kay Thompson and O. L. Mahlman. (20) May 17.
History and Legal Phases of the Smelting-Smoke Problem. Ligon Johnson. (Paper presented before Am. Inst. of Min. Engrs.) (16) Serial beginning May 19.
The Cascade Flotation Machine.* Clifford R. Wilfley. (16) May 19.
Alloys of the Non-Ferrous Metals. W. M. Corse and G. F. Comstock. (Abstract of Paper read before Steel Treating Research Club of Detroit.) (47) May 25.
Electric Iron Smelting. J. O. Boving. (22) May 25.
Petrography as an Aid to Flotation.* Donald G. Campbell. (16) May 26.
Leaching of Low-Grade Copper Ores.* Joseph Irving. (16) May 26.
Steel and Steel Alloys. G. N. Pingriff. (19) May 26.
The Media Mill, Webb City, Mo.* H. B. Pulsifer. (Abstract from *Bulletin*, Am. Inst. of Min. Engrs.) (16) May 26.
Sintering Flue Dust at Mingo Junction.* H. V. Schiefer. (20) May 31.
Notes Upon the Hydrometallurgical and Electrolytic Treatment of Zinc Ore. E. E. Watts. (Paper read before Am. Chemical Soc.) (105) June 1.
Chemical Reactions of Iron Smelting. Walter Mathesius. (Paper read before Am. Iron and Steel Institute.) (105) June 1; (20) June 21.
Alloys of Chromium, Copper, and Nickel. (47) June 1.
Cyaniding at Comacaran Mines, Central America.* William M. D'Arcy. (16) June 2.
New Tramway at the Tacoma Smeltery.* W. C. Kuhn. (16) June 2.
Metallurgy of Ferromanganese. Robert J. Anderson. (Abstract from *Iron Trade Review*.) (16) June 2.
Non-Reversing Regenerative Furnace For Copper Smelting.* Walter G. Perkins. (103) June 2.
Mayari and Nickel Steels Compared. S. W. Parker. (20) June 7.
Electric Resistance Furnace.* (Abstract of Paper read before Inst. of Metals.) (20) June 7.
Electric Process For Small Steel Castings. R. F. Flintermann. (Abstract of Paper read before Am. Electrochemical Soc.) (47) June 8; (22) June 8; (20) May 10.
Ferro-Manganese in the Iron and Steel Industry. Robert J. Anderson. (Abstract of Communication to the *Journal* of the Franklin Inst.) (47) June 8.
Progress of the Electric Steel Industry. John A. Mathews. (Abstract of Paper read before the Am. Electro-Chemical Society.) (47) June 8; (20) May 10; (22) June 8; (26) June 22.
Pneumatic Concentrator and Amalgamator.* Frank A. Stanley. (16) June 9.
Theory of Ore Flotation.* H. P. Corliss and C. L. Perkins. (103) June 9.
Impurities in Electrolytic Copper Refining.* Lawrence Addicks. (105) June 15.
Notes on Electric Steel Melting. J. L. Dixon. (Abstract of paper read before Am. Electrochemical Soc.) (22) June 15.
The Bethlehem 10-Ton Girod Steel Furnace. C. A. Buck. (Abstract of Paper read before Am. Electrochemical Soc.) (22) June 15.

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Metallurgical—(Continued).

The Rennerfelt Electric Furnace. C. H. Vom Baur. (Abstract of Paper read before Am. Electrochemical Soc.) **(22)** June 15; (20) May 17; (26) June 29.
Laws of Distribution of Lead in Southeast Missouri Ores. Sergio Bagnara. (16) June 16.
Volumetric Chromate Determination of Lead. John Waddell. (103) June 16.
American Smelting and Refining Company's Tests With Sulphur and Sulphuric Acid on Soils.* P. J. O'Gara. **(103)** June 16.
Carbon Monoxide Dangers at Iron Works. **(20)** June 21.
Crude-Oil Melting Furnace.* (Abstract of Paper read before Inst. of Metals.) (20) June 21.
Open-Hearth Furnace of Large Capacity.* **(20)** June 21.
The Metallurgy of Ferrosilicon.* Robert J. Anderson. (Abstract from *Iron Trade Review.*) **(16)** June 23.
The Heat Treatment of Large Forgings.* William Beardmore. (Abstract of Paper read before Inst. of Mech. Engrs.) **(20)** June 28.
The Empirical Formula in Milling Control. A. J. Sale. **(103)** June 30.
An Ore-Testing Laboratory. Carl J. Trauerman. **(16)** June 30.

Military.

Papers on Munitions, with Bibliography, read before the Spring Meeting of the American Society of Mechanical Engineers. **(55)** May; **(20)** Serial beginning May 10.
Road Work in Mexico With the Punitive Expedition.* James A. O'Connor. **(100)** May-June.
Triton Electroplating Plant at Washington Barracks, D. C. R. W. Crawford. **(100)** May-June.
Relation of the Gas Industry to Military Needs. J. H. Burns. (Paper read at Annual Convention of the Pennsylvania Gas Assoc.) **(83)** May 1.
Silent Warfare.* Nicholas Flamel. **(19)** May 12.
A Mining Engineer's Experience in the Artillery.* T. A. Rickard. **(103)** May 12.
The Gage Problem in Rifle Manufacture. John Q. Tilson. **(20)** May 17.
The Blind Sufferers From the War, and Their Future Employment. C. Arthur Pearson. **(29)** May 18.
Motor Traction in Modern War.* Victor W. Page. **(46)** May 19.
United States Munitions: 3 to 6-in. Cartridge Cases.* **(72)** May 24.
Military Engineering.* Albert K. Dawson. **(46)** May 26.
The Equipment of an Army. **(19)** May 26.
United States Munitions: The Springfield Model 1903 Service Rifle.* **(72)** Serial beginning May 31.
Preparedness. A. M. Lockett. **(55)** June.
Electrical Communication in the Field.* **(26)** June 1.
The Mathematics of Warfare.* J. Malcolm Bird. **(19)** June 2.
The Modern Automobile Torpedo.* Edward F. Chandler. **(46)** June 2.
The More Recent Applications of Electricity in the Present War, Especially in the Treatment of Diseases and Wounds Arising Therefrom.* W. Leeming. **(26)** June 15.
Naval and Military Searchlights.* Charles Osmond Knowlton. **(72)** June 21.
Transportation of Military Traffic.* **(15)** June 22.
Sixteen Cities For the New National Army. **(20)** June 28.
La Fabrication des Principales Matières Explosives et l'Utilisation des Produits Résiduaires. J. Meunier. **(33)** Serial beginning Apr. 28.
Nouvelles Usine Vickers pour la Fabrication des Mitrailleuses.* **(33)** June 16.
Batteries Américaines de Gros Calibre Mobile sur Voies Ferrés.* **(33)** June 16.
Ein artilleristiches Problem. F. Bützberger. **(107)** May 5.

Mining.

Further Notes on Safety-Lamps.* Simon Tate. **(106)** Mar.
The "Chalmers-Black" Non-Accumulative Visual Indicator for Shaft-Signalling.* John B. Thomson. **(106)** Mar.
The Cementation Process as Applied to Mining (François System). Thomas Blandford. **(106)** Mar.
Principles of Coal Washing. Henry Louis. (Paper read before Northern Section of the Coke Oven Managers' Assoc.) **(57)** Apr. 5.
Surface Subsidence Due to Mining Operations.* **(11)** Apr. 13.
Installation and Maintenance of Underground Plant.* W. Matthew Baird, Jr. (Paper read before Assoc. of Mining Engrs.) **(22)** Apr. 13.
Some Practical Notes on the Economical Use of Timber in Coal Mines.* F. C. Lee. (From paper read before the North of England Inst. of Mining and Mech. Engrs.) **(57)** Apr. 20.
Sources of Oil Supply. John Thompson. (Read before National Assoc. of Colliery Managers.) **(22)** Apr. 27.

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Mining—(Continued).

Philippine Coals and Their Use. F. R. Yeasiano. (57) April 27.
Mine Gas. W. W. Chernitzyn. (Abstract.) (57) Serial beginning April 27.
Some Practical Notes on the Economical Use of Timber in Coal Mines.* F. C. Lee. (106) May.
Electric Supply to Collieries.* G. S. Corlett. (Paper read before the Manchester Geological and Min. Soc.) (106) May; (47) April 27.
Observations on the Order of Working the Coal Seams in the North of England. Wm. D. Harbit. (22) Serial beginning May 4.
Pumping Potash from Nebraska Lakes.* R. P. Crawford. (16) May 5.
Concentrating Canadian Molybdenite. H. H. Claudet. (Paper read before Canadian Mining Inst.) (16) May 5.
Operation of the Yukon Placer Act.* C. A. Thomas. (16) May 5.
Recent Developments of the Whiting Hoist in Deep Winding.* B. Gray and J. Whitehouse. (Abstract from *Journal* of South African Inst. of Engrs.) (57) May 11.
Mine Water as Boiler Feed. Edwin M. Chance. (Abstract from *Coal Age.*) (57) May 11.
Anaconda Copper Mining Co. (16) May 12.
Mining Methods in Great Britain.* (Abstract from *Mine and Quarry.*) (45) May 12.
Coal Mining in the Transvaal. C. C. Smith. (45) May 12.
Righting a Wrecked Gold Dredge.* Lewis H. Eddy. (16) May 12.
The Kirkland Lake Gold District.* J. C. Bateman. (103) May 15.
Accident Prevention in Quarrying. (86) May 16.
Electric Winding at Victoria Colliery, Ebbw Vale.* (26) May 18.
The Future of the Iron and Coal Industries in Normandy.* (22) May 18.
Nevada Consolidated Copper Co.* (16) May 19.
Picturesque Side of Jerome.* A. J. Hoskin. (16) May 19.
Working Dirty Beds of Coal.* Rowland Gascoyne. (45) May 19.
The Anthracite Situation. William Griffith. (45) May 19.
Amortization and Depreciation: A Discussion. W. P. Sleeman. (103) May 19.
Henry C. Perkins, and the Cost of Mining: An Interview.* T. A. Richard. (103) May 19.
Molybdenum in the Hualpai Mountains.* L. Webster Wickes. (103) May 19.
Use of Explosives in Quarrying Shale. C. B. Willis. (Paper read before Inst. of Paving.) (76) May 22.
New Method for Mining Bituminous Coal in Connellsville, U. S. A.* Patrick Mullen. (22) May 25.
Deep Creek, Clifton Mining District, Utah.* A. E. Custer. (16) May 26.
Drill-Sharpening.* (103) May 26.
Spontaneous Combustion in Coal Mines. Rowland Gascoyne. (45) May 26.
The Romance of Refrigeration and Its Application to Mining.* Andrew D. Brydon. (Abstract of Paper read before Nat. Assoc. of Colliery Mgrs.) (45) May 26.
Tipple at Powhatan in Pocahontas Field.* Henry J. Edsall. (45) May 26.
Rotary Dump Installed Underground.* E. L. Berger. (45) May 26.
Magnetic Segregation and Ore Genesis. Joseph T. Singewald, Jr. (103) May 26.
Sampling an Erratic Orebody. L. A. Parsons. (103) May 26.
Nickel in Ontario. (29) Serial beginning June 1.
Colliery Managers' Examination Papers.* (22) June 1.
Quarrying Limestone Underground.* Raymond B. Ladoo. (16) June 2.
Concreting a Creek Channel.* C. W. Wardle. (16) June 2.
Anthracite Coal Strippings Near Scranton and Wilkes-Barre, Penn.* Thomas F. Kennedy. (45) June 2.
Some Theories on Mine Subsidence. J. F. Kellock Brown. (45) June 2.
The Extralateral Right: Shall It Be Abolished? William E. Colby. (103) June 2.
The Pazña Tin-Mining District, Bolivia.* Francis Church Lincoln. (103) June 2.
Mining Problems on the Rand.* H. Foster Bain. (103) Serial beginning May 26.
Grade Revision For Underground Haulage. R. D. Brown. (57) June 8.
The Belmont Camp, Nevada.* Wilson W. Hughes. (16) June 9.
The Mining Industry of Peru. F. C. Fuchs. (16) June 9.
Occurrence and Utilization of Antimony Ores. (Abstract from *Bulletin* of the Imperial Inst.) (16) June 9.
Siberian Mine-Timbering Methods.* Henry M. Payne. (16) June 9.
Double-Range System of Speed Control For Adjustable-Speed Induction Motors.* F. B. Crosby. (45) June 9.
Drilling and Shooting Coal. F. C. Pick. (45) June 9.
Revival of Mining at Marysville, Montana.* L. S. Ropes. (103) June 9.
Perseverance Mine Powder-Thawer.* D. J. Argall. (103) June 9.
The Higher Training of Managers. G. L. Kerr. (Abstract of Paper read before Mining Inst. of Scotland). (22) June 15; (57) June 15.

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Mining—(Continued).

Methods of Mining in the Pennsylvania Anthracite Field. Hugh M. Crankshaw. (Abstract of Paper presented to Manchester Geological and Mining Soc.) (22) June 15; (57) June 15.
Cannop Colliery and Its Water Difficulties. Jno. J. Joynes. (Paper read before the Forest of Dean Branch of the Nat. Assoc. of Colliery Managers.) (22) June 15.
Acetylene Mine Lamps.* William Maurice. (Abstract of paper read before Inst. of Min. Engrs.) (22) June 22; (57) Serial beginning June 15.
The Clarification of Mill Water. G. Nicolai. (Abstract from *Metal und Erz.*) (16) June 16.
Disregard of Danger the Cause of Mountain King Fatalities. Lewis H. Eddy. (16) June 16.
"Caliche" Deposits of Atacama Desert, Chile.* Fred MacCoy. (16) June 16.
How Best to Eliminate the Bigger Mine Accidents.* Frank Haas, (Abstract of Paper read before West Virginia Coal Mining Inst.) (45) June 16.
Anaconda's Finances.* W. R. Ingalls. (16) June 16.
Mining: The Great Adventure. T. A. Richard. (Address delivered before Colorado School of Mines.) (103) June 16.
The Miami Appeal: Majority Opinion of the U. S. Circuit Court of Appeals, Philadelphia. (103) Serial beginning June 16; (16) Serial beginning June 23.
Lump-Coal Storage and Reclaiming Plant* H. M. McFarland. (45) June 23.
California Dredge With Four Tailings Conveyors.* Lewis H. Eddy. (16) June 23.
Manitoba as a Mining Province.* J. A. Campbell. (16) June 23.
The Black Oak Mine.* W. H. Storms. (103) June 23.
Automatic Skips in Shaft With Rope-Guides.* H. Vincent Wallace. (103) June 23.
Utilizing Coal Mine Waste in the Pacific North West. Henry Hull. (57) June 27.
Dangers Accompanying Use of Carbide.* J. R. Allardyce. (45) June 30.
Saline Valley Tramway.* F. C. Carstarphen. (Abstract from *Proceedings*, Am. Soc. C. E.) (103) June 30.
Underground Churn Drilling.* Guy W. Bjorge. (45) June 30.
Mining in Southern Peru.* Francis Church Lincoln. (16) June 30.
Antimony Deposits of Arkansas.* Ellsworth H. Shriver. (103) June 30.
The Amorphous Silica of Southern Illinois.* E. A. Holbrook. (16) June 30.

Miscellaneous.

The Law and The Engineer. George H. Montgomery. (5) Jan.-June, 1916.
How Can the Engineer Improve His Public Standing? F. W. Hanna. (96) Mar. 22.
Graphical Calculus. Charles A. Ellis. (4) April.
Education in Engineering. D. S. Jacobus. (Paper read before Am. Soc. of Mech. Engrs.) (8) April.
The Decimal System of Coinage, Weights and Measures. Harry Allcock. (Abstract of a lecture before Inst. of Civ. Engrs.) (104) Serial beginning April 20.
The Organisation of Engineering. Michael Longridge. (Abstract of paper read before Inst. of Mech. Engrs.) (22) April 27; (26) May 4; (73) May 11.
How to Determine Efficiency. George H. Shepard. (9) May.
Cause and Prevention of Industrial Casuality.* H. Weaver Mowery. (9) May.
Constructive Public Policy for Utility Growth. John A. Britton. (111) May 1.
Assistance in Developing Foreign Fields.* E. E. Pratt. (111) May 1.
Education and Training of Engineering Apprentices. H. A. Bennie Gray. (Abstract of paper read before Soc. of British Gas Industries.) (66) May 1.
Sand Devastation.* Percy Collins. (19) May 5.
Initiation of Explosions. Walter Arthur. (Paper delivered before Am. Chemical Soc.) (19) May 5.
Works Organization and Efficiency: Discussion. W. Ripper. (29) May 11; (47) May 18.
The Physiological Role of Calcium in Plants. Thérèse Robert. (Abstract). (19) May 12.
Chemical Control in the Leather Industry. David Quick Hammond. (19) May 12.
Precision in Chemical Weighing. William Norman Rae and Joseph Reilly. (19) Serial beginning May 5.
Gravel Deposits: Their Origin and Economic Development.* Wm. Artingstall. (86) May 16.
The Human Factor in Industry.* Luther D. Burlingame. (72) Serial beginning May 17.
Measurement of Reflection Factor.* M. Luckiesh. (27) May 19.
The Complexity of the Chemical Elements. (11) May 25.
The Recent Industrial and Economic Development of Indian Forest Products. R. S. Pearson. (29) Serial beginning May 25.
System in a City Engineer's Office. Manley Osgood. (60) June.

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Miscellaneous—(Continued).

The Brownian Movement of Electrified Particles in Gases. A. Schidlof. (19) Serial beginning May 26.
The Application of the Modern Theory of Wages. Felix Bayle. (22) June 1.
The Fundamental Principles of Good Lighting. P. G. Nutting. (Abstract from *Journal* of the Franklin Inst.) (66) June 5.
Engineers Must Coöperate for Recognition and Service. (Address at Annual Convention of Am. Assoc. of Engrs.) Gardner S. Williams. (13) June 7.
Shall Great Britain and America Adopt the Metric System? W. R. Ingalls. (Abstract of paper read before Inst. of Min. and Metallurgy.) (73) June 8.
Chemicals for Laboratory Use. William Rintoul. (Paper read before Glasgow Section of Society of Chemical Industry.) (19) June 16.
Detecting the Pretense of Deafness.* Robert Foy. (Abstract of paper in *La Nature.*) (19) June 16.
The Engineer in Politics. Robert G. Dieck. (Abstract of Paper in *Journal* of Oregon Soc. of Engrs.) (86) June 20.
Time Studies for Delay Allowances in Rate Setting.* Dwight V. Merrick. (72) June 21.
Wages for Ability, Output and Service.* W. E. Freeland. (20) June 28.
The Human Potential in Industry. Otto P. Geier. (55) July; (20) Serial beginning May 31.
Organisation Scientifique de l'Usinage.* P. Denis. (33) Serial beginning April 14.
La Rééducation des Avengles.* Lucien Fournier. (33) May 5.

Municipal.

Municipal Work in Rangoon From 1907 to 1916.* Launcelot Paul Marshall. (114) May.
Small-Town Electric Service Problems.* (27) May 5.
Municipal Engineering in America. A. F. Macallum. (Abstract of paper read before Ottawa Branch of Canadian Soc. of C. E.) (104) June 15.
Presidential Address before Inst. of Municipal and County Engrs. Philip H. Palmer. (104) June 29.

Railroads.

Notes on the Working of a Rack Railway. William Theodore Lucy. (63) 1915-1916, Pt. II.
Design of Passenger Terminals.* J. L. Busfield. (5) Jan.–June, 1916.
Grade Separation—Two Distinct Methods.* Samuel T. Wagner. (Delivered before Engineers' Soc. of Penn.) (98) Nov.-Dec., 1916.
The Federal Valuation of Railways. Towson Price. (8) April.
Doctors of Transportation. Geo. A. Post. (36) April.
American Coaling Stations for Locomotives.* George Frederick Zimmer. (11) April 13.
The Watford New Electric Train Service.* (12) April 13.
Railway Water Supply.* C. R. Knowles. (65) May.
The Locomotive Firebox and Combustion Chamber. J. T. Anthony. (Abstract of paper read before the Southern and South-western Railway Club.) (47) Apr. 20.
Tunnels.* (21) Serial beginning May.
The Chambers Regulator.* (21) May.
Enlarging a Busy Tunnel under Traffic.* (87) May.
Reclaiming Battered and Worn Rails.* John Reinehr. (87) May.
Special Track Work in Paved Streets.* H. F. Heyl. (87) May.
Boiler Patches for Locomotives.* M. J. Cairns. (25) May.
McClellon Water-Tube Firebox.* (25) May.
Straightening Surface Bent Rails.* J. Rodman. (87) May.
New Power for the Lehigh Valley.* (25) May.
Air Brake Lever Computations.* Lewis K. Sillcox. (25) May.
Steel as a Material for Locomotive Fireboxes. R. P. C. Sanderson. (Abstract of paper read before Inst. of Locomotive Engrs.) (21) Serial beginning May.
Box Car Side Door.* (25) May.
Turbine for Locomotive Drive.* Victor W. Zilen. (64) May 1.
Pennsylvania Changes at Baltimore under Discussion Again.* (13) May 3.
Construction of a Comprehensive Low-Grade Line.* (15) May 4.
The Resawing of Lumber.* D. C. Curtis. (15) May 4.
Rules for the Mobilization of Freight Cars. (15) May 4.
Canadian Commission Against Public Ownership. J. L. Payne. (15) May 4.
Canadian Commission Minority Report. (15) May 4.
Origin and Development of the Railway Rail in England and America.* G. P. Raidabaugh. (Abstract of paper presented to Iron and Steel Inst.) (22) May 4.
Pennsylvania Locomotive Brick Arch Tests.* (15) May 4.

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Railroads—(Continued).

Slack Action in Long Passenger Trains; Its Relation to Triple Valves of Different Types and Consequent Results in the Handling of Trains. J. A. Burke and William Hotzfield. (Abstract of paper read before Air Brake Assoc.) (18) Serial beginning May 5.

Handling Heavy Passenger Trains on Grades with Air Brakes Exclusively. J. E. Fitzgerald. (Abstract of paper read before Air Brake Assoc.) (18) May 5.

Safe Life of Air Brake Hose. M. E. Hamilton. (Paper read before Air Brake Assoc.) (18) May 5.

Determination of Actual Sectional Errors in Railroad Track Scales.* (18) May 5.

A 120-Tons Capacity Gondola Car for the Virginian Railway.* (18) May 5.

Railway Enquiry Commission's Report (Canada). Sir Henry L. Drayton, W. M. Acworth and Alfred Smith. (96) Serial beginning May 10; (18) Serial beginning May 5.

The Government White Elephant at Quebec.* C. V. Johnson. (96) May 10.

When to Ship Freight by Motor Truck and When by Rail? C. C. Williams. (13) May 10.

Canadian Northern Electrification at Montreal.* W. C. Lancaster. (15) May 11.

Functional Interrelation between the Component Parts of the Air Brake System.* W. E. Dean. (Paper read before Air Brake Assoc. Convention.) (15) May 11; (18) June 2.

Pennsylvania Track Elevation at Johnstown.* (15) May 11.

High-Pressure Electric Railway Systems. (26) May 11.

The Choice of Voltage for Railway Electrification on the Direct-Current System.* F. Lydall. (73) Serial beginning May 11.

The Rate Advance Case. (Abstract of Hearings before the Interstate Commerce Commission.) (18) Serial beginning May 12; (15) Serial beginning May 18.

Methods and Cost of Changing 17 Miles of Railroad Track From Narrow Gage to Standard Gage.* Henry R. Somes. (86) May 16.

The Highest Railroad Shop in the United States.* Frank A. Stanley. (72) May 17.

Offer Divers Solutions of Canada's Railway Problem. (13) May 17.

Theory, Practice and Results of Fuel Economy. W. P. Hawkins. (15) May 18; (25) June; (18) May 19.

New Union Station-Plants Completed for St. Paul.* (15) May 18; (13) June 7.

Government Ownership in Foreign Countries. W. M. Acworth. (15) May 18; (18) May 12.

New Automatic Block Signals on the Southern Railway.* (18) May 19.

Would Depress New York Central and Elevate Lackawanna Through Syracuse.* (13) May 24.

Locomotive Feed Water Heating.* C. M. Moderwell. (Report at Railway Fuel Assoc. Convention.) (15) May 25; (25) June; (18) May 19.

Pattern for Locomotive Cab Window Shield.* (101) May 25.

The Julian-Beggs Automatic Stop and Train Control System.* (18) May 26.

Mountain and Santa Fe Type Locomotives for the Southern Railway.* (18) May 26.

Refrigerator Cars for the Pacific Fruit Express Co.* (18) May 26.

Progress in the Rail Problem Marked by Sharp Differences in View.* (13) May 31.

D. C. Track-Circuits. (21) June.

Flexible Firebox Stays.* (21) June.

Developments in Railway Shop Tools. (25) June.

Organization and Methods for Handling Rods.* Ernest A. Miller. (25) June.

Centralized Production of Locomotive Repair Parts.* George Armstrong. (25) June.

Electrical Equipment at a Great Western Goods Station.* (26) June 1.

Suggestions for the Conservation of Fuel. (15) June 1.

Double Deck Stock Cars for the Santa Fe.* (15) June 1; (25) June;* (18) June 9.

To Increase Transportation Efficiency. Howard Elliott. (15) June 1.

The German Railway Record in China. (11) June 1.

New Kansas Interurban Line Serves Central Cities.* (17) June 2.

Mechanics of the Chilled Iron Wheel. George W. Lyndon and F. K. Vial. (Abstract of joint address before Railway Club of Pittsburgh.) (18) June 2; (51) July 27; (25) May.

Rail Failure Attributed to Track Service.* (20) June 7.

"Hold-Main" Signals on the L. & N.* (15) June 8.

The Pennsylvania's New Electric Locomotive.* (15) June 8; (17) June 9.

Modern Train Despatching—Some Needed Forms. Harry W. Forman. (15) June 8.

Mikado and Consolidation Types Compared.* (Discussion based on Testing Plant *Bulletin No. 28* of Penn. R. R. Co.) (15) June 8; (25) June.

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Railroads—(Continued).

The New High-Speed Three-Phase Locomotives of the Italian State Railways.* P. Verole and B. Marsili. (Abstract from *Rivista Tecnica delle Ferrovie Italiane.*) (73) June 8.
Up-State New York Lines Need Relief. Thomas Conway, Jr. (17) Serial beginning June 9.
Successful Use of Steel Trolley Wire By Pacific Railway.* S. H. Anderson. (17) June 9.
Economics of the Supply Train. D. D. Cain. (18) June 9.
A Seventy-Ton Capacity Well Car for the Pennsylvania R. R.* (18) June 9.
Train Handling. G. H. Wood. (Paper read before Car Foremen's Assoc. of Chicago.) (18) June 9.
Making Locomotive Driving Boxes.* Frank A. Stanley. (72) June 14.
Pennsylvania Locomotive of the Decapod Type.* (15) June 15.
Powdered Coal. (Abstract of Committee Report at Convention of International Railway Fuel Assoc.) (15) June 15; (18) May 19.
Wheel Shop Practices, Minneapolis, St. Paul & Sault Ste. Marie Ry., Minneapolis, Minn.* (18) June 16.
Shop Fire Protection. Paul Hevener. (Paper read before Car Foremen's Assoc. of Chicago.) (18) June 16.
Steel Car Shop, Louisville & Nashville R. R., South Louisville, Ky.* Millard F. Cox. (18) June 16.
The Fuel Situation as Affected by the War. (15) June 22.
The Railroads of Canada and the War.* (15) June 22.
The Maintenance of Way Labor Problem.* (15) June 22.
Methods of Increasing the Train Load. (15) June 22.
Increasing Track Capacity by Signaling. (15) June 22.
Heavier Car Loading Would Eliminate Car Shortage.* (15) June 22.
Electricity as an Aid to Railroad Operation. (15) June 22.
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PAPERS AND DISCUSSIONS

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PAPERS AND DISCUSSIONS

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AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

THE THREE 15-CUBIC YARD DIPPER-DREDGES, GAMBOA, PARAISO, AND CASCADAS, AS SUPPLIED AND USED ON THE PANAMA CANAL

By Ray W. Berdeau, Jun. Am. Soc. C. E.

To be Presented September 19th, 1917.

Synopsis.

The object of this paper is to place before the Society the result of the writer's study of the design, operation, and efficiency of the three large 15-cu. yd. dipper-dredges supplied by the Bucyrus Company for use on the Panama Canal.

The 15-cu. yd. dipper-dredges, *Gamboa* and *Paraiso,* were requisitioned by the Isthmian Canal Commission as part of the permanent equipment of the Panama Canal and for immediate use in completing the channel through Gaillard Cut (formerly Culebra Cut). A contract was made with the Bucyrus Company, which stipulated that the dredges were to be ready for towing to the Isthmus on December 1st, 1913, and January 1st, 1914. The *Gamboa* was accepted at Port Richmond, N. Y., on February 16th, and the *Paraiso,* on April 13th, 1914. The *Gamboa* reached the Isthmus on March 16th,

Note.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings,* and, when finally closed, the papers, with discussion in full, will be published in *Transactions.*

and was placed in operation on April 4th, 1914, being followed by the *Paraiso,* which arrived on May 22d, and started work on June 7th, 1914. The total cost of the two dredges, and the towing, etc., was $573 287.40, to which initial cost should be added $3 092.50 for the *Gamboa,* and $1 786.21 for the *Paraiso,* the respective amounts necessary to place them in commission after their arrival.

These dredges operated so efficiently that the Isthmian Canal Commission placed another contract with the Bucyrus Company for a third dredge, of improved design, called the *Cascadas,* which was accepted at Port Richmond, N. Y., and successfully towed to the Isthmus, where it arrived on October 21st, 1915, practically 2 months ahead of the promised delivery, and was placed at work in Gaillard Cut on October 31st, 1915, at a total cost of $376 180.

GAMBOA AND PARAISO

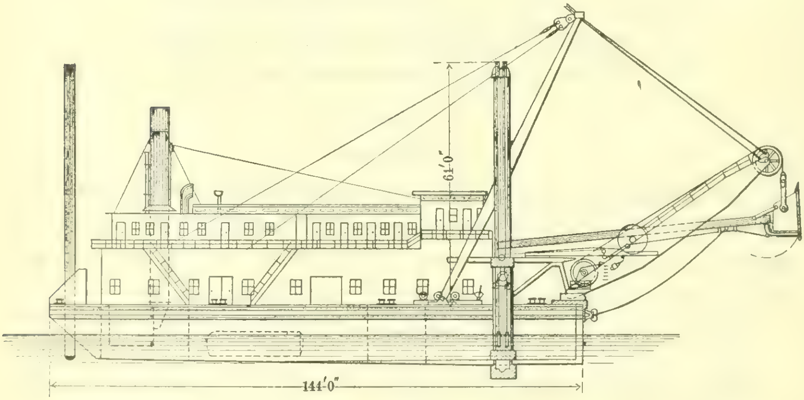


FIG. 1.

The following are the principal dimensions, etc., of the *Gamboa* and *Paraiso*:

Length of hull	144 ft. 0 in.
Beam, moulded	44 " 0 "
Depth, moulded	13 " 6 "
Draft	8 " 0 "
Digging depth, below water line	50 " 0 "
Displacement	1 730 tons.

One main engine, two cylinders, compound, 16 by 28 by 24 in.

One swinging engine, two cylinders, compound, 12 by 16 in.

One backing engine, two cylinders, compound, 12 by 16 in.

Two forward spud engines, two cylinders, compound, 12 by 16 in.

One stern spud engine, two cylinders, 9 by 9 in.

Two deck winches, two cylinders, 6 by 6 in.

Two boilers, Scotch marine type, 126 in. diameter, 138 in. long, water pressure 150 lb.

Two forward spuds, 48 by 48 in., and 82 ft. long.

One stern spud, 30 by 30 in., and 83 ft. 6 in. long.

Swing circle, 24 ft. in diameter.

Bail pull, 235 000 lb.

Hoisting pull on spud rope due to engine, 88 000 lb.

"Pin up" pull on single cable, with brake on engine, 160 000 lb.

Capacity of rock dipper, 10 cu. yd.

Capacity of mud dipper, 15 cu. yd.

Capacity of fuel oil tanks, 14 200 gal.

The displacement of the *Cascadas* is 2 095 tons, and the hull is 144 ft. long, 55 ft. beam, and 15½ ft. deep. Thus, it is 11 ft. wider than the others, making less reactions on the spuds, less metacentric variation when digging over the sides, and it allows the spuds to be inset. The spud-well construction differs from that of the *Gamboa* and *Paraiso,* as their forward spuds are placed outside of the hull, with tapering sponsons fore and aft to transmit the reactions to the sides of the hull.

Buckets.—The dredges were supplied with interchangeable buckets of two sizes, one with a capacity of 15 cu. yd. and another of 10 cu. yd., for use in rock excavation. Having been placed in Gaillard Cut in rock digging exclusively, the larger dippers have been seldom used; the smaller ones, as supplied by the contractors, were of extra massive construction, but were of insufficient strength to withstand the severe use and the impact from a dipper stick load of 131 000 lb., and were replaced later by the Missabe type of cast manganese-steel dippers. The over-all dimensions of the new dipper are 10½ by 9 by 9 ft.; the lips are 3¼ in. thick at the bottom bands, and the body consists of a front and back casting with lap-riveted joints at the sides; and, in addition, the lip is a separate casting riveted to the front piece and joined thereto by the rivets of the tooth ribs. Recently, the back and bottom of this dipper has been further reinforced, and the dipper

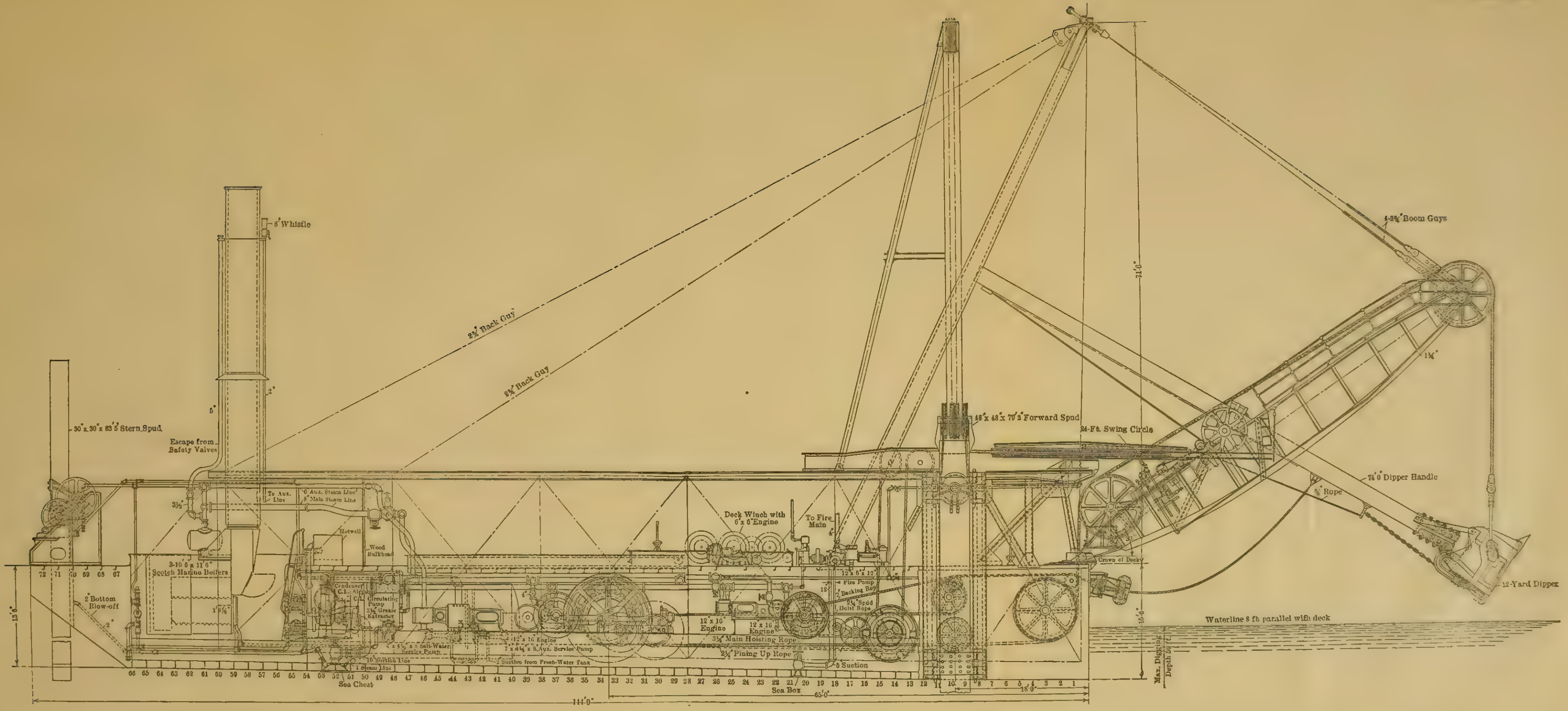
is expected to give greater service and satisfaction than the preceding models.

Cables.—The dippers are hung on a 275-ft., $3\frac{1}{4}$-in., extra pliable, improved, plow-steel wire, center cable consisting of six strands of 37 wires each, intended to withstand a bail pull of 235 000 lb., the minimum cable life being 3 days and the maximum 35 days. The rapid deterioration of the cables is due to the severe abrasion they receive while at their deeper digging depths (from 35 to 50 ft.), coupled with the deteriorating effects of their constant travel over an undersized point of boom sheave, which is grooved for a wire rope $3\frac{1}{4}$ in. in diameter and is about 8 ft. in diameter at the bottom of the groove. The sheaves are of cast steel, with long, heavy cast-steel hubs, bronze-bushed, to distribute the pressure over the 11-in. sheave pins, pressed into place; the cables, which are the largest in use on dipper-dredges, are hardly satisfactory for the service required, for which the supplying manufacturers refuse to guarantee them.

Dipper Handles.—The dipper handles on all the dredges are 72 ft. long, over all, and are reinforced top and bottom with 2 by 12-in. bars and by 1 by 22-in. plates on both sides of each dipper stick. Long-leaf yellow pine is used in the construction of the sticks, and white oak for dead wood. The racks are manganese-steel castings, with a pitch of 3 in., and are about 12 in. wide. They are shrouded to the pitch line, the top of the shrouded portion being ground to form a rolling surface for similar shrouding on the rack pinions. Heavy steel castings are used to connect the dipper hinge frame, and similar castings are used to connect the dipper back braces, which are securely bolted to the end of the handle by a large number of $2\frac{1}{2}$-in., horizontal and vertical through bolts. The weight of the dipper handle is 81 000 lb., its life averaging about 6 months, that of the rebuilt handles being 3 months.

Saddle-Block.—The innovation in the design of the saddle-block has proved as useful as it is interesting. The slide plate is separated into two parts for assembling on the flanged shipper-shaft, leaving a passage in the middle, which permits the main hoist cable to run in a straight line, from the foot to the point of the boom sheave. This eliminates the usual hump sheave, which is generally placed near the upper end of the boom and is necessary on other dredges to lift the single-hoist cable clear of the saddle-block.

PLATE VII.
PAPERS, AM. SOC. C. E.
AUGUST, 1917.
BERDEAU ON
DIPPER-DREDGES
ON THE PANAMA CANAL.



The heavy unit construction facilitates the guiding and holding of the dipper handle in a much more secure manner, and improves the meshing of the racks on the dipper handles with the pinions of the 9½-in. hollow, nickel-chrome-steel, shipper-shaft, at times to such an extent that the teeth are stripped from them both. By building a heavier bucket, the dipper handle, saddle-block, shipper-shaft, and hoisting arrangement offer opportunity for improvement in design, in that the dipper stick would be stronger if it was made of one piece, two main hoisting cables being used, running on each side of the dipper handle, thereby increasing the life of the hoisting cable and also the dipper stick. This dipper stick at times becomes bowed, and, due to the sliding fit with the saddle-block, necessitates immediate re-alignment; this could be obviated if a rolling fit was presented, the rollers being supplied with bearings under compression. The shipper-shaft bearings, which are bolted to the top chord of the boom, project so that the flanges of the brake wheels, which are built of steel castings 75 in. in diameter with a 12-in. face and bolted to the flanges on each end of the shipper-shaft, engage the bearing boxes when lifted for removal, making an extended operation of changing the shipper-shaft. The brakes are of the double-acting type, and are actuated by a steam thrust-cylinder, the steam valve of which is controlled by a floating lever and operated by a hand-lever on the craneman's platform.

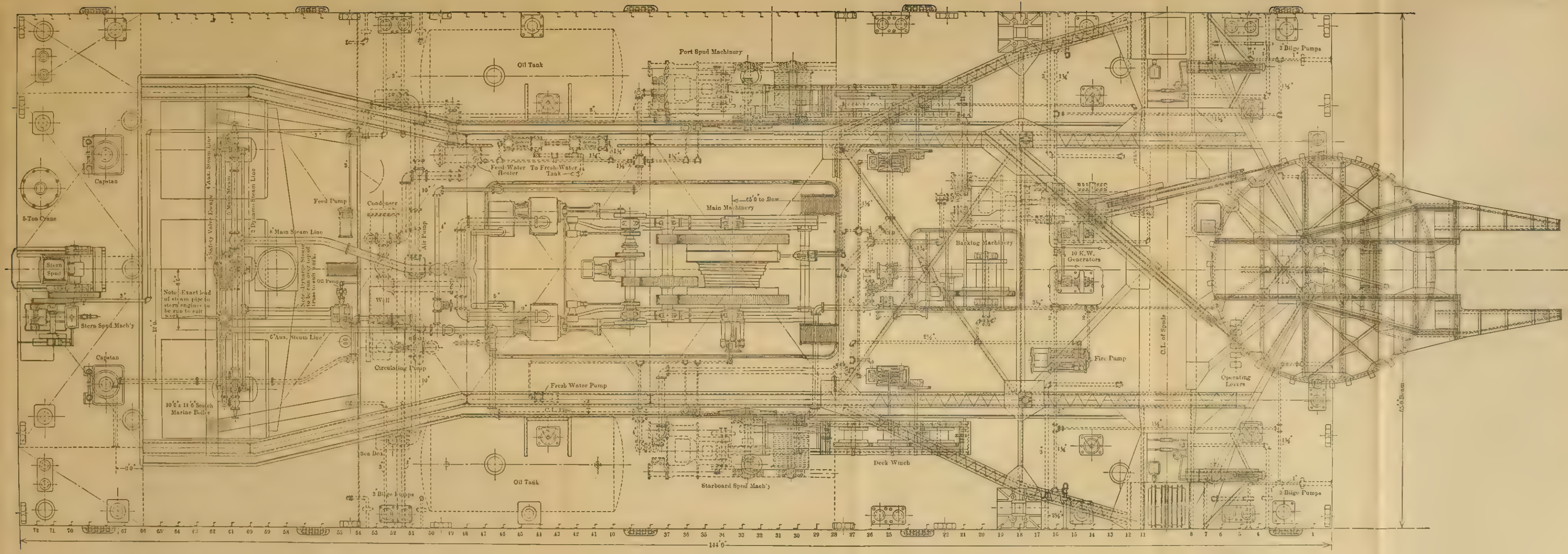
Booms.—The booms on these dredges are 62 ft. long, of the plate-girder type, with curved top and bottom chords. All parts of these booms are supposed to be of ample section to withstand developed stresses, which, due to the heavy type of work, have been such as to necessitate reinforcement of the different booms that, with complete machinery, weigh 113 000 lb. They are equipped with a steam-operated boom brake, steam shipper-shaft brakes, and a steam dipper-trip, the cylinder of which is mounted above the foot of the boom, and is connected with the latch bar on the dipper by an endless wire rope, the circuit beginning at the upper end of the dipper handle, leading around the sheave on the stand just below the shipper-shaft, and thence around the sheave attached to the cross-head of the steam dumping cylinder, which permits dumping in any position. The boom feet are of heavy steel castings, with webs and flanges of such length as to permit adequate riveting. The boom is stepped into a steel casting pivot, formed with sockets to receive the boom feet; the pivot

rotates on a heavy cast-steel base plate, securely bolted to the hull, with its flanges extending over the front of the hull. The pintle is bronze-bushed, and has a bronze wearing plate or washer between the boom step collar and the base plate, and another bronze bushing and wearing plate is used between the base plate and the center casting of the swing circle.

Main Engines.—The main engines are specified to work at a steam pressure of 135 lb. while condensing. They are of the horizontal, twin-tandem, compound type, with 16 and 28-in. cylinders and 24-in. stroke, mounted on heavy self-contained cast-iron bed-plates of the Tangye pattern. The crank shaft is of forged steel, 12 in. in diameter, with journals 9¼ in. in diameter and 14 in. long, and with screw-adjusting bearings. The connecting rods, valve stems, and the adjustable bronze-shoed cross-heads are of steel, finished and arranged for taking up wear. The link motion and reverse gears are omitted on the *Cascadas,* and are replaced by a steam turning gear, comprising a steam reversing engine geared to the crank shaft with a releasing jaw clutch operated from the engine-room. The low-pressure cylinders have piston valves working in renewable cast-iron valve cages, a stuffing-box being incorporated between the high and low-pressure cylinders. The eccentric bearings of the *Cascadas* are larger than those of the *Gamboa* and *Paraiso,* and the *Cascadas* is equipped with an overhead 15-ton traveling crane.

Hoisting Drum and Gears.—The hoisting drum is of the differential type of cast steel, and is bushed with bronze. The small diameter is 69 in. and the large diameter is 84 in. at the bottom of the grooves, which are for 3¼-in. wire cable. The drum is mounted loose on the 16-in., forged-steel, main hoisting shaft, having journals 11¾ in. in diameter and 18 in. long. Power is applied to the drum by two outside, wood-lined, band frictions, one on each side of the drum, both operated by a single steam cylinder 14 in. in diameter, placed at one end of the drum shaft and attached to a thrust spindle passing through the center of the shaft. The drum is of the usual type supplied by the Bucyrus Company on its dredges, and has a barrel-like shape, which permits the maximum digging force and the slowest speed when the dipper is excavating and the angle between the hoisting rope and the dipper handle is sharpest; as the rope is wound on the larger diameter, the dipper is hoisted, allowing any desired increase

PLATE VIII.
PAPERS, AM. SOC. C. E.
AUGUST, 1917.
BERDEAU ON
DIPPER-DREDGES
ON THE PANAMA CANAL.



of hoisting speed when the maximum bail pull is not required. The diameter of the drum could be increased, which would reflect in length of cable life. The drum is driven by two heavy spur-gears, 12 ft. in diameter, one on each side, meshing with the corresponding pinion of the intermediate shaft. The gear hubs next to the bearings are lined with bronze. The intermediate shaft is driven from the crank shaft through a single gear, which has steel castings rim-bolted to a heavy cast-steel spider arranged for rim replacements without stripping the shaft. The intermediate gear rim is split for easy removal and replacements, and, like the pinion on the crank shaft, has cut teeth. The intermediate shaft is bolted directly on the engine bed-plates; the bed-plate also contains the drum-shaft bearings, and is of cast steel, forming an extension of the engine bed-plate, and is securely bolted to it and the structural base built into the hull.

Swinging Circle Machine and Guide-Sheaves.—The swinging circle is of structural steel, 24 ft. in diameter, and mounted on top of the hull truss; connection is made to the boom with two heavy built-up girders, extending out from the circle, one on each side of the boom. The center is a heavy steel casting, securely bolted to the circle, and has an I-beam rim reinforced with ½-in. plates, the jaws being fastened to the boom at the forward ends. Changes have been effected in the rope anchorage of the dredges by making the swing rope around the circle in four separate pieces with open socket connections, which renders complete stripping unnecessary when the rope is changed. Two 42-in. cast-steel sheaves, on top of the hull truss, grooved for a 2-in. rope, and complete with shaft bearings and 4⅜-in. sheave pins, are supplied for guiding the swing rope from the circle to the swinging drum. The swinging machinery is operated by an independent double engine, with 12-in. cylinders and 16-in. stroke, and reversing link gear. The engine and drum are mounted on heavy structural steel bases built into the hull, and the links are reversed by the steam thrust-cylinder controlled by the lever which operates the throttle.

Backing Engine.—The backing engine and drum are mounted on a structural steel base built into the hull, and the drum is operated by a separate 12 by 16-in., double, non-reversing engine. The cast-steel drum (26 in. pitch diameter) is driven by an outside-band friction clutch, actuated by a steam thrust-cylinder, 5 in. in diameter, and carries a 2-in. steel rope. The diameter of this drum is too small,

as it breaks the strands of the cable, prevents proper reeling, and causes slack on the drum, which prohibits uniform backing of the bucket. The gear reduction between the crank and the 7-in. drum shaft is single, and a band brake prevents the running out of the rope.

Forward Spud Machinery.—Each forward spud is operated by an independent, double, 12 by 16-in. engine, with link motion reverse, which is operated by a steam thrust-cylinder controlled by the lever which operates the throttle valve. The spud drums are 42 in. in diameter and are grooved for 2¼-in. wire rope. The *Cascadas* is "pinned up" and the spuds are lifted by a 2¼-in. wire rope passing around the four sheaves at the top of each spud; the pinning-up ropes are similar to those on the *Gamboa* and *Paraiso,* but the spud hoist ropes run over sheaves which are on a gantry mounted near the spud casings and extending above the highest position of the spuds.

The Bucyrus Company has developed a big improvement in this design, as it dispenses with the sheaves at the lower end of the spuds, where the wire ropes are quickly cut by the sharp stones found in the rock excavation in "the Cut." Wood-lined band-brakes, operated by a steam cylinder, are supplied to hold the dredge when "pinned-up", and the spud tackle rope is taken up by four parts of rope. Each spud engine is connected to its drum by double reduction gearing of cast steel; and a suitable friction clutch, operated by a steam thrust-cylinder, is provided to disconnect the drum from the engine, allowing the dredge to rise and fall with the tide when not in use. The spud drum machinery and engines are supported by a heavy structural steel frame built into, and set far enough from the side of, the hull to permit ready access to the spud machinery on the outboard side of the foundation. The sheaves guiding the spud ropes to the drum have a pitch diameter of 45 in., are cast steel, bushed with bronze, and have annealed, forged-steel, sheave pins. The stern spud machinery is arranged for a trailing spud that is hoisted by rack and pinion, through two gear reductions, by a 9 by 9-in., double engine, placed on and operated from the deck, at the stern of the dredge. The intermediate shaft is fitted with a band-brake, to hold the spud in its proper position, and carries a jaw clutch for disconnecting the drum from the shaft and the engine. The stern spud is of structural steel, 30 by 30 in. and 83 ft. 5 in. long, and is held in place by a sliding-fit arrangement somewhat similar to that

of the saddle-block. This creates the necessity of immediate re-alignment when the spud becomes slightly bowed. The forward spuds are of structural steel, 48 by 48 in. and 72 ft. long, with 8 by 8 by $\frac{3}{4}$-in. corner angles, $\frac{3}{4}$-in. side plates, and suitable diaphragms, those of the *Cascadas,* as mentioned, having all sheaves at the top of the spud. As heavy as they are, these spuds have to be removed every 90 days, three being broken in as many days, in one case.

Deck Fittings.—On the main deck, outside of the house, there are two three-drum winches, which are used for moving scows, and are equipped with internal driving engines. Each engine is of the horizontal type, and has two 6-in. cylinders with 6-in. stroke; the throttle-valve is in the main steam chest, and also acts as a reversing valve. The drums are steel castings, bushed with bronze, with 24-in. barrels 12 in. long, and equipped with friction clutches of the outside-band type, as well as band-brakes. There are ten double mooring bits, ten single deck spools, and sixteen deck chocks.

Boilers and Fittings.—Steam is provided in the *Cascadas* by a battery of three boilers of the Scotch marine type, arranged so that any two boilers can be used at one time, the third one being a spare, permitting of blowing down without tying up the dredge, this being an additional boiler to the two supplied, respectively, to the *Gamboa* and *Paraiso.* The boilers are constructed for a working pressure of 150 lb. per sq. in.; each is supplied with two Morrison suspension furnaces, and is equipped with marine pop safety-valves, stop valves, pressure gauges, blow-off cocks, etc. The stack is double, and about 50 ft. high above the tops of the boilers. One surface condenser is supplied, having approximately 1 500 sq. ft. of cooling surface, together with an independent air pump and a 10-in. brass-runner, centrifugal, circulating pump, driven by a separate engine. One 300-h.p., feed-water heater of the closed type and two 7 by $4\frac{1}{2}$ by 8-in. brass-fitted, horizontal, duplex, boiler feed-pumps are provided, also a similar pair for general service, one fresh-water, $4\frac{1}{2}$ by $2\frac{3}{4}$-in., horizontal pump, one 12 by 6 by 12-in., horizontal, duplex, brass-fitted pump, and a bilge pump for each compartment. The piping system is very complete, each line having a stop-valve to cut it off from the boiler or main, so that repairs can be made at any time without shutting down the system.

All the large pipes are flanged, and the steam fittings for 200 lb. pressure and exhaust fittings for 125 lb. are extra heavy.

Tanks.—There are four steel fresh-water tanks in the stern of the hull; two of 30 000 gal. capacity are supplied for boiler-feed purposes, on the port and starboard sides, respectively. There is an 8 000-gal. tank on the port side for galley use and a duplicate tank on the starboard side for ballast and trimming. There are two 18 000-gal. oil tanks in the hull forward of the boiler, one on each side of the dredge, and two oil-feed pumps are used to supply oil to the boilers.

Electric Light Plant.—Two 10-k.w., 110-volt, direct-current generators, each connected directly to a vertical, automatic engine, are provided for lighting the dredge, operating the house lights, the searchlight, and the lights used in night dredging.

House and Cabin Construction.—The main deck house is 108 ft. long and 41 ft. wide from the front end to Frame 48 (about 95 ft.), and 30.4 ft. from Frame 48 to the after end. The height is about 17 ft. for a distance of 18 ft. from the bow end, and 15 ft. throughout the remaining length. The upper deck over the house is 112 ft. long and 44 ft. wide throughout, projecting 6 ft. 9 in. over the lower deck house on all sides. Quarters are supplied for 11 "gold" (white) American employees and 57 "silver" (colored) employees. Three officers' staterooms have hot and cold-water service, one berth and single wall lockers. All other staterooms are fitted with double, wooden, built-in berths, with two drawers and lockers under each lower berth, double, wooden wall lockers, and wash-basins with hot and cold water. The galley and dining-room are amidships, between the "gold" and "silver" quarters; in the latter, thirty-eight standee bunks with canvas bottoms, and thirty-eight sanitary steel lockers are supplied. There is also separate modern bath and toilet service on the main deck for both "gold" and "silver" men.

The Hull.—The hull of each dredge is of steel. The *Cascadas* is 144 ft. long over all, 55 ft. wide, and has a mean depth of 14 ft. 6 in. (15 ft. 6 in. at bow at center, and 13 ft. 6 in. at stern at center), with a straight taper fore and aft. The deck has a 6-in. camber made by straight lines from the center to each side.

Operation.—All three dredges are 15-cu. yd. machines, built by the Bucyrus Company, in the United States, and have been working until recently in Gaillard Cut of the Panama Canal. The material

TABLE 1.—MONTHLY PERFORMANCE OF DREDGE *Gamboa.*

Date.	Cubic yards.	Cost of operation.	Cost of maintenance.	Total cost.	Cost per yard.
1914.					
Apr	32 805	$4 446.82	$10 798.18	$15 245.00	$0.4647
May	108 185	7 365.53	16 165.11	23 530.64	0.2175
June*	123 199	6 764.68	24 279.25	31 043.93	0.2519
July	108 896	7 441.27	21 148.06	28 589.33	0.2625
Aug	121 850	7 002.58	14 772.77	21 775.35	0.1787
Sept	111 355	6 561.27	12 238.25	18 799.52	0.1688
Oct	137 060	7 377.96	9 015.69	16 393.65	0.1205
Nov	140 905	7 612.19	8 902.44	16 553.81	0.1174
Dec	166 092	8 147.03	12 997.82	21 144.85	0.1274
1915.					
Jan	178 370	7 823.27	14 351.67	22 147.94	0.1244
Feb	149 554	6 308.69	8 939.04	15 247.73	0.1020
Mar	207 870	7 425.32	9 643.36	17 068.68	0.0821
Apr	168 725	7 003.25	10 593.31	17 596.56	0.1043
May	165 720	7 599.38	11 839.11	19 438.49	0.1173
June*	168 725	7 075.09	11 332.41	18 407.50	0.1091
July	171 370	7 376.09	9 421.10	16 797.19	0.0980
Aug	199 425	8 102.46	12 111.33	20 213.79	0.1014
Sept	280 550	8 263.33	8 455.81	16 719.14	0.0596
Oct	249 515	7 610.10	14 612.47	22 222.57	0.0891
Nov	260 845	8 280.17	10 431.38	18 711.55	0.0717
Dec	266 470	8 877.27	9 624.75	18 502.03	0.0694
1916.					
Jan	232 855	7 515.06	8 590.39	16 105.45	0.0692
Feb	293 230	9 103.00	10 056.95	19 159.95	0.0653
Mar	304 006	9 241.50	10 790.25	20 031.75	0.0695
Apr	263 275	8 896.18	10 460.12	19 356.30	0.0735
May	271 235	8 638.56	8 996.67	17 635.23	0.0650
June*	304 450	9 127.41	12 003.65	21 131.06	0.0694
July	320 190	9 448.95	9 055.02	18 503.97	0.0578
Aug	129 210	6 221.29	8 046.99	14 267.78	0.1108
Sept	150 175	7 748.87	8 238.84	15 987.71	0.1063

YARDAGE EXCAVATED BY FISCAL YEARS.

Date.	Cubic yards.	Cost of operation.	Cost of maintenance.	Total cost.	Cost per yard.
July 1st, 1913, to July 1st, 1914.					
	264 189	....	....	$69 819.57	$0.3021
July 1st, 1914, to July 1st, 1915.					
	1 825 122	....	....	233 190.41	0.1278
July 1st, 1915, to July 1st, 1916.					
	3 097 226	....	....	226 586.01	0.0731
July 1st, 1916, to October 1st, 1916.					
	599 575	....	....	48 759.46	0.0713

* End of fiscal year.

TABLE 2.—MONTHLY PERFORMANCE OF DREDGE *Paraiso.*

Date.	Cubic yards.	Cost of operation.	Cost of maintenance.	Total cost.	Cost per yard.
1914.					
June*	69 812	$9 002.69	$11 458.21	$20 460.90	$0.2931
July	82 203	5 717.11	10 221.48	15 938.59	0.1939
Aug	95 475	6 728.67	20 155.38	26 884.05	0.2816
Sept	105 470	7 236.62	17 889.63	25 126.25	0.2382
Oct	125 605	7 380.00	9 644.65	17 024.65	0.1355
Nov	139 916	7 938.98	15 110.26	23 091.19	0.1650
Dec	144 165	7 824.00	12 371.43	20 195.43	0.1401
1915.					
Jan	176 862	8 111.45	11 380.86	19 492.31	0.1102
Feb	172 057	5 617.64	9 310.49	13 928.13	0.0868
Mar	134 030	6 395.02	8 740.37	15 135.39	0.1129
Apr	204 250	7 410.48	9 274.15	16 684.63	0.0817
May	184 510	7 239.92	10 889.73	18 129.65	0.0983
June*	174 685	7 880.58	8 855.95	16 766.53	0.0960
July	170 060	8 003.89	9 970.81	17 974.70	0.1057
Aug	203 815	8 064.34	10 748.12	18 812.46	0.0923
Sept	220 940	7 280.87	10 986.89	18 267.76	0.0827
Oct	291 675	8 859.52	15 502.24	22 361.76	0.0767
Nov	262 925	8 740.54	9 193.33	17 933.87	0.0682
Dec	312 920	9 716.54	10 153.78	19 870.32	0.0635
1916.					
Jan	195 515	7 295.94	8 587.49	15 883.43	0.0812
Feb	236 235	8 710.18	11 975.35	20 685.53	0.0876
Mar	299 155	9 099.70	11 323.76	20 423.46	0.0683
Apr	232 365	8 887.67	10 152.90	19 040.57	0.0819
May	306 435	8 649.36	10 871.43	19 520.79	0.0637
June*	272 064	9 115.15	11 275.58	20 390.73	0.0749
July	233 930	6 418.34	7 192.01	13 610.35	0.0581
Aug	315 915	9 823.62	12 505.83	22 329.45	0.0714
Sept	268 250	8 921.49	9 029.42	17 950.91	0.0689

YARDAGE EXCAVATED BY FISCAL YEARS.

July 1st, 1913, to July 1st, 1914:					
	69 812	....	....	$20 460.90	$0.2931
July 1st, 1914, to July 1st, 1915:					
	1 739 228	....	....	228 396.80	0.1313
July 1st, 1915, to July 1st, 1916:					
	3 004 104	....	....	231 165.38	0.0769
July 1st, 1916, to October 1st, 1916:					
	818 095	....	....	53 890.71	0.0658

* End of fiscal year.

TABLE 3.—MONTHLY PERFORMANCE OF DREDGE *Cascadas.*

Date.	Cubic yards.	Cost of operation.	Cost of maintenance.	Total cost.	Cost per yard.
1915.					
Oct.	695	$427.10	....	$427.10	$0.6145
Nov	296 280	9 371.94	$10 056.25	19 428.19	0.0656
Dec	321 065	9 961.47	10 221.35	20 182.82	0.0628
1916.					
Jan.	292 675	9 054.96	10 769.15	19 824.11	0.0677
Feb	330 605	8 982.54	9 939.51	18 922.05	0.0573
Mar	309 125	8 796.39	10 723.06	19 519.45	0.0632
Apr	246 786	8 649.02	10 775.53	19 424.55	0.0787
May	316 770	8 848.07	10 818.53	19 666 60	0.0621
June*	286 491	9 171.57	9 995.13	19 166.70	0.0669
July	340 185	9 383.79	10 244.25	19 628.04	0.0577
Aug	208 967	8 135.09	9 481.83	17 616 92	0.0862
Sept.	122 504	6 864.80	5 367.74	12 232.54	0.0987

YARDAGE EXCAVATED BY FISCAL YEARS.

July 1st, 1915, to July 1st, 1916.

	2 400 492	....	....	$156 561.57	$0.0651

July 1st, 1916, to October 1st, 1916.

	666 656	....	.. ..	49 477 50	0.0742

* End of fiscal year.

excavated consisted of hard and soft rock, to a depth of from 35 to 47 ft. When the scows are moored on the port side of the dredge, their loading is sometimes handicapped by the inadequate view obtained by the operator while he works, as direct vision of the mid-scow pockets is obstructed by the structural frame supporting the boom and the port forward spud; these pockets, when empty, necessitate the lowering of the bucket into the pocket, so that the spoil when dumped, will not damage the bottom doors. An arrangement of an ordinary mirror, about 24 by 14 in., on the operator's right helps to a certain extent, but, as the scow in the view does not appear quite natural, the scow strong backs suffer. Reflecting mirrors could be arranged to give a direct full view of the loading section, thereby enabling quicker operation. The *Gamboa* and *Paraiso* assisted in raising the drill barge, *Teredo*—which was blown up and sunk in the channel at the foot of Cucaracha Slide on July 20th, 1914—by attaching the main hoist cables to slings and raising the wreck. Tables 1, 2, and 3 give the

yardage of the respective dredges, *Gamboa, Paraiso,* and *Cascadas,* and the material placed in scows alongside the dredges. The accompanying costs include operation, that is, wages of crew, subsistence of crew, fuel, and lubricants, maintenance, that is, the cost of keeping the equipment in first-class physical condition, and depreciation only. Extra heavy 10-yd. manganese-steel dippers were used on this work, the dredges working continuously in three 8-hour shifts, under the charge of the Resident Engineer, W. G. Comber, M. Am. Soc. C. E., and the Superintendent of Dredging, Mr. James Macfarlane.

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PAPERS AND DISCUSSIONS

THE DISTRIBUTION OF STRESSES IN MITERING LOCK-GATES, WITH SPECIAL REFERENCE TO THE GATES ON THE PANAMA CANAL

By Henry Goldmark, M. Am. Soc. C. E.*

Synopsis.

This paper is based on an extended study of the laws which govern the distribution of stresses in different parts of a mitering lock-gate under service conditions. For reasons mentioned in the paper, the problem is indeterminate, so that the solution was based on the theory of elastic work.

A general method of computation is first outlined, which is simple in principle, though the computations are somewhat laborious. It is believed to be more complete and accurate than any previously developed.

An application to several of the lock-gates on the Panama Canal follows. The resultant loads which are sustained by the different horizontal girders and by the vertical bracing are computed in detail, as well as the pressures exerted by the gates against the bottom sills.

* This paper will not be presented for discussion at any meeting, but written communications on the subject are invited for subsequent publication in *Proceedings* and with the paper in *Transactions*.

Some practical conclusions, drawn from the results obtained, are also given, as a guide for future designs.

General.

Mitering gates have been used in canal locks for several centuries, and are still the preferred form, to the virtual exclusion of other types. In large modern locks, they are generally built of steel, and are among the most expensive structures used in hydraulic works. In the interest of economy as well as safety, therefore, careful designing, based on a correct determination of the stresses, is a matter of importance.

The problem of finding the stresses in such gates is intricate, and has been the subject of much investigation in the past. In connection with the design of the gates for the Panama Canal, the largest yet built, the writer had occasion to make a rather detailed study of the subject. It seems proper to put on record some of the results obtained, especially as American literature on lock-gates is very scanty. It is believed that the method of calculation used is novel, and is an advance on previous practice.

The stresses are of two kinds, those due to the weight of the gate leaf, and those due to water pressure. The former do not affect the design of the gate seriously, except in the parts which serve to support it on the foundation, and to connect it to the anchorage in the walls. They are greatest when the lock chambers are emptied to permit of inspection and repairs. Their computation is quite simple when the weight of the metal work is known.

The stresses due to hydrostatic pressure, on the other hand, govern the dimensions of the principal members, as well as most of the secondary parts. The external loads in this case, too, are entirely definite, as well as their points of application, but the stresses they produce are somewhat indeterminate. This is due, in large part, to the complex structure of the leaf, which has numerous horizontal and vertical members riveted together at their intersections and covered by a continuous sheathing. The primary function of the different members, in transferring the hydrostatic load to the masonry, is well defined, but the rigid connections cause the deflection and, hence, the stress in any

one member to depend, not only on its direct load, but also on other parts of the gate.

The distribution of the stress, therefore, is complicated, even in single-leaf gates which span the lock at right angles and support the water pressure by beam action.

In miter-gates, which act as arches, there are further difficulties. The first arises from variations in the surface of contact on the miter and quoin-posts, where the gates bear against each other and against the hollow quoin. Especially if timber bearing pieces are used, the exact point at which the reaction acts, is uncertain, so that it is necessary, in the computations, to assume it to have a considerable deviation from the center of the bearing, in order to obtain the maximum stresses.

A further difficulty arises from uncertainty as to the pressure exerted by the gate against its sill. In single-leaf gates this will not change appreciably after the first adjustment is made. In miter-gates, however, there are wide variations in the sill reaction at different times, with corresponding changes in the stresses. This is due to changes in the length of the leaves arising from variations in temperature, wear in the contact pieces on the gate posts and sills, and from various minor causes. Even with careful workmanship and fitting, it is hardly possible to ascertain the sill pressure exactly, although, as will be shown later, it is feasible to arrive at limiting values, not likely to be exceeded in practice.

In this paper, a method of calculation is developed, by which the sill reaction and the loads on the different horizontal and vertical girders may be found in a given gate for an assumed hydrostatic loading and various conditions of sill contact. After these loads have been fixed, the stresses in the different members can be computed without special trouble. The gates are assumed to be of steel and "horizontally framed" with numerous horizontal girders. In the gate with "vertical framing", which has only two main horizontals, the stresses are more determinate, and the theory is much simpler.

The problem is first stated in general terms, and a solution is found applicable to gates of varying dimensions, girder spacing, and cross-sections. It is then applied to the largest and the smallest of the

Panama gates. Finally, some practical conclusions derived from the computations are given, as a guide in future designs.

Statement of Problem.—A mitering lock-gate consists of two leaves, which together support the water pressure due to the difference of head on the opposite sides of the gate.

If there is no contact at the bottom sill when the gate is closed and under pressure, the entire hydrostatic load is transferred by arch action to the side-walls. On the other hand, if the leaves bear against the sill, the latter will carry a part of the load, the remainder being, as before, supported by the lock walls.

The proportion of the total load which will go to the sill, in any given case, depends on the structural arrangement of the gate frame and the relative adjustment of the gate and sill.

The object of the investigation given herewith was to determine, for the Panama lock-gates, the pressure of the gates against the sills for different adjustments, the distribution of the loading between the several horizontal girders, and the stresses in the vertical framing.

Previous Investigations.—The determination of the laws which govern the distribution of loading in the horizontal and vertical members of mitering-gates has been made the subject of extended studies by several distinguished engineers in the past.

M. Chevallier* made what was perhaps the first study of this subject. It included a series of tests with wooden models. The conclusions he drew from his experiments, as to the interaction of horizontal and vertical members in gate leaves, are in entire accordance with subsequent investigations, and of much interest even now. He gave no general formulas or rules applicable to the large gates of modern times.

In 1867, M. Lavoinne† published a mathematical investigation covering the same subject. His method is very complicated, although applicable only to gates having equal horizontals spaced at equal vertical distances. Owing to these assumptions, and for other reasons, Lavoinne's formulas are not applicable to large modern gates, in which the cross-section and spacing vary from the top to the bottom of the leaf.

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* *Annales des Ponts et Chaussées* for 1850.

† *Annales des Ponts et Chaussées.*

In 1887, M. Galliot* presented a new mathematical study, making practically the same assumptions as Lavoinne, but attaining somewhat simpler results.

In his treatise on "Mitering Lock Gates", published in 1892, First Lieut. (now Brig.-Gen.) H. F. Hodges, U. S. A., gave a discussion on vertical framing on a somewhat different basis, and deduced valuable rules for its practical dimensioning.

Method of Calculation Used in the Present Investigation.—The method herewith presented was developed by the writer in a less perfect form, in 1899, in a study on lock-gates made for the Board of Engineers on Deep Waterways.

The solution is based on the well-known "method of least work." It takes account of irregular spacing in the horizontal girders, as well as of variations in their cross-sections, and considers the cross-bending as well as the direct compression in the girders. In case timber is used for cushions at the miter and quoin posts, or at the sill, the formulas obtained can easily be modified, so as to allow for the difference of material.

Although the method is correct in theory, the unavoidable lack of homogeneity in the steel, the difficulty of determining the vertical and horizontal rigidity of the leaf exactly, still more the uncertainty as to the relative adjustment of the gate leaves and the sill, prevent a very close determination of the actual stresses.

It is believed, however, that the results obtained are reliable within reasonable limits, and will prove of much use in analyzing the strength and stiffness of existing gates or proposed designs. It should be added that the formulas when applied to the gates of the Poe Lock, at Sault Ste. Marie, gave results agreeing quite closely with deflection measurements made by the writer.

Like most applications of the elastic theory to complex structures, the method of least work cannot precede, but must follow the complete design. In other words, it is necessary to adopt a detailed arrangement of all parts and afterward determine the distribution of the stresses in the different members.

Effect of Vertical Stiffness.—A gate leaf consisting only of a certain number of horizontal girders and an absolutely flexible sheathing would have no vertical stiffness. Such a gate would transfer no load

* *Annales des Ponts et Chaussées.*

to the sill, except the water pressure which acts on the lower half of the bottom panel and is carried directly to the sill by the sheathing. Each horizontal girder would support simply the load which corresponds to the hydrostatic head due to its position.

In practice it is impossible and undesirable to build a gate without vertical rigidity. The sheathing, combined with the quoin and miter posts and the intermediate vertical brace frames and intercostals, forms a vertical girder of considerable strength. By its resistance to bending, this girder modifies the loads on the different horizontals, making them greater or less than those corresponding to the hydrostatic pressure. As a rule, the vertical girder transfers a part of the total water pressure to the sill.

For purposes of calculation, the leaf is taken as consisting of a horizontal and a vertical system of girders crossing each other at right angles. The horizontal system consists of the several main girders or arches, spaced as they are in actual construction. The vertical system is assumed to be equivalent to a single girder extending continuously over the whole length of the leaf from the quoin to the miter post. Its flanges are formed by the gate sheathing, and its web is equivalent in total cross-section to the web plates in the several vertical frames and end posts. This simplification is justified by the close spacing of the vertical frames and intercostals, which prevents the skin plates from buckling.

Sill Contact.—In case there is no contact at the sill, even when full water pressure acts, the entire load, of course, will be carried by the horizontal arches to the side-walls.

If, in such a gate, all the horizontals are proportioned to support the hydrostatic head with exactly the same unit stresses in the steel, they will all have exactly the same deformations under load. There will be no bending stress in the vertical girder, so that it will remain straight, even after the gate is supporting the water pressure. The loads on the several horizontals will be those corresponding to their hydrostatic head.

In practice, the horizontals near the top are always stronger than theory requires, in order to ensure increased safety against accidental blows and to avoid the use of unduly small rolled shapes. It is also hardly possible to design all the other horizontals so that they shall sustain exactly the same unit stresses. There will always be some

variation, therefore, in the deflections of the different horizontal girders. This will produce a tendency to bend the vertical girder which is rigidly connected to the horizontals, and its resistance to bending, in turn, will affect the deflections and modify the loads of the horizontal frames.

However, the effect of vertical rigidity, when there is no sill contact, will be very small, except at the extreme top of the leaf.

The case of "no contact" should always be provided for in the design, for, from various causes, it is likely to occur in all mitering lock-gates as either a temporary or permanent condition.

In ordinary cases there will always be a greater or less reaction at the sill. If the water on the up-stream side extends to the top of the gate, and there is no lower pool, the greatest sill pressure theoretically possible would be equal to two-thirds of the total load supported by the gate. This maximum can only occur when the adjustment is so inaccurate that, even with continuous contact along the sill, the two leaves will touch only at the very top of the miter posts, even when the gate is subjected to the full head of water. This is an extreme case which would seriously overstrain the gate, and can be avoided by ordinary care in adjusting the leaves and sill.

A much smaller reaction may be counted on as a practical maximum. It seems quite safe to assume what is sometimes called "perfect contact", that is, continuous contact along the quoin and miter posts and also along the sill when the gate is closed, but before it is subjected to water pressure.

With both timber and metallic bearings, the actual conditions will probably correspond to lower sill pressures, as there will rarely be absolute sill contact in the dry.

Therefore, two conditions of adjustment at the sill were considered in the computations:

(1) No contact at the sill, even with full head;

(2) Simultaneous contact at the sill, miter, and quoin posts, before the water pressure is applied.

Let Fig. 1 represent the vertical section and Fig. 2 the plan of a gate leaf, of length, L, consisting of $(n + 1)$ horizontal arches and a continuous vertical girder, the stiffness of which is assumed to be uniformly distributed over the length of the leaf. The sheathing is

supposed to carry the water pressure directly to the horizontals, and the connections to be such as to permit the transference of horizontal reactions between the arches and the vertical girder at their intersections. Let the magnitude of these reactions be denoted by X_0, X_1, ... X_n per linear horizontal unit of leaf. They will be both positive and negative in direction, and will act normally against the arches exactly as water pressure does.

If, further, P_0, P_1, ... P_n, are the direct water loads on the several arches per linear unit, their resultant total loads will be:

$$(P_0 + X_0)\ L\ (P_1 + X_1)\ L\ .\ .\ .\ .\ (P_n + X_n)\ L.$$

With no contact at the sill, all these loads are carried by arch action to the hollow quoins; but, if the lowest arch bears against an absolutely fixed sill, that arch will carry no load to the side-wall, and $(P_n + X_n)\ L$ will become the sill reaction.

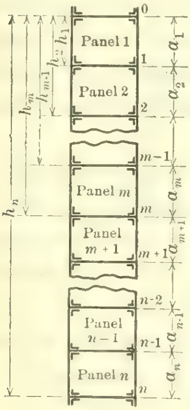


FIG. 1.

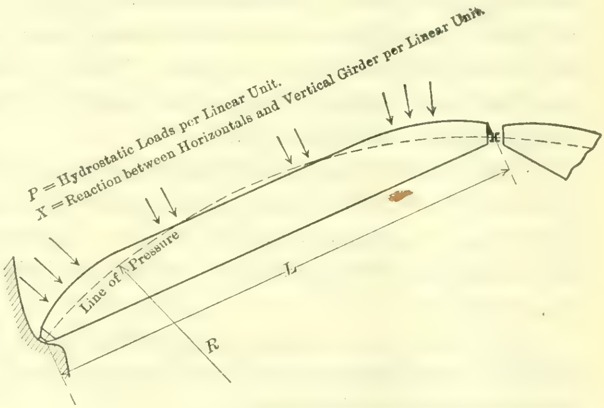


FIG. 2.

In any case the only forces acting on the vertical girder will be the transverse loads,

$$X_0\ L,\ X_1\ L,\ .\ .\ .\ X_n\ L.$$

The static conditions of equilibrium as applied to this girder give only two equations:

$$\Sigma\ X = X_0 + X_1 + \ .\ .\ .\ .\ X_{n-1} + X_0 = 0\ .\ .\ .\ .\ .\ .\ .\ .\ .\ .\ .\ .\ .\ .\ .\ .\ .\ .\ (1)$$

$$\Sigma\ M = X_0\ h_n + X_1\ (h_n - h_1) + \ .\ .\ .\ X_{n-1}\ (h_n - h_{n-1}) = 0\ .\ .\ .\ .\ (2)$$

for determining the values of X_0 . . . X_n, although there are always at least three such unknown quantities.

Similarly, if the gate as a whole is considered, the reactions of the horizontals against the side-walls are indeterminate, as their number is in excess of the number of equations which can be obtained from static conditions.

The stresses in indeterminate structures of this kind, however, can be found by an application of the method of least work. The principle on which this depends may be briefly explained as follows:

If a perfectly elastic structure is subjected to external forces, the fibers in its parts will be deformed until a new condition of equilibrium is reached. The work done in this deformation (the internal work or elastic potential) will always be equal in amount to the external work, that is, the work done by the external forces.

By the principle of least work, the reactions and internal stresses corresponding to the new condition of equilibrium, in addition to being consistent with the statical equations, must be such as will make the total internal work done by the structure, in passing from its original to its new condition, a minimum.

This principle is sometimes derived from the theory of virtual displacements, but may almost be considered axiomatic, representing the theory of equilibrium in its most general form as applied to elastic solids.

The application of the method of least work to this problem consists in stating the work of internal deformation in terms of the known external loads and the indeterminate quantities X_0, X_1 . . . X_n, and finding those values of the latter, which, besides satisfying the static Equations (1) and (2), will give a minimum value for the total internal work done, while the gate passes from a condition of no stress to that in which the full water pressure is supported.

The internal work in the whole leaf will consist of the following parts:

(1) That due to arch action in the horizontal girders, involving generally both direct compression and cross-bending; and

(2) The work of bending the vertical girder.

The work done as the result of the shearing stresses in the arches and the stresses in the web members of the vertical girder, being relatively quite small, may be neglected.

The bending in the horizontal girders is due to the eccentricity of the line of pressures or resultants with reference to the center of gravity at the different cross-sections. For arches with continuous curvature this eccentricity is quite small, but occurs to some extent in all gates.

If U_h = the internal work in the horizontal, and U_v that in the vertical girders, then,

$$U = U_h + U_v \text{ is the total internal work.}$$

For any elastic solid under purely axial stress (that is, direct compression or tension), the work of deformation will be:

$$U_s = \int_0^L \frac{T^2}{2\,E\,F}\,d\,l \qquad (3)$$

and, for one subject to cross-bending only (beam action),

$$U_b = \int_0^L \frac{M^2}{2\,E\,I}\,d\,l \qquad (4)$$

in which equations:

L = the total length of the member;
F and I = the cross-sections and moments of inertia at any point;
E = the modulus of elasticity; and,
T and M = the total axial force and bending moment at any cross-section.

From Equations (3) and (4) a general expression for the work equation for all parts of a mitering lock-gate (such as is shown in Fig. 1) may be written:

$$U = \sum_{m=0}^{m=n}\left[\int_0^L \frac{T_m^{\,2}}{2E\,F_m}\,d\,l + \int_0^L \frac{M_{hm}^{\,2}}{2\,E\,I_{hm}}\,d\,l\right] + \sum_{m=1}^{m=n}\int_0^{a_m} \frac{M_{vm}^{\,2}}{2\,E\,I_{vm}}\,d\,l \qquad (5)$$

In this equation, m corresponds to any horizontal arch and also to the panel of the vertical girder just above the arch so denoted. The axial thrust and bending moment at any point in a horizontal are represented by T_m and M_{hm}, respectively; and M_{vm} is the bending moment at any point in the vertical girder. The first two terms give U_h (work in all horizontal arches), the integration being for the length of each individual arch and the summation to include the

$(n+1)$ different arches. The last term gives U_v (work in vertical girder), the integration being for each separate panel, while the summation includes all of them.

The cross-sections, F_m, and the moments of inertia, I_{hm} are the average values for any given arch, and I_{vm} is taken as uniform for any given panel of the vertical girder.

As will be shown later, T_m, M_{hm}, and M_{vm}, and, hence, U (Equation 5) may be readily expressed in terms of the indeterminates, $X_0 \ldots X_n$, and the known quantities. If, in the resulting equation, the condition of statical equilibrium is introduced by putting, for X_n and X_{n-1}, their values in terms of the other $(n-1)$ variables (using Equations 1 and 2), U will be expressed as a function of $X_0 \ldots X_{n-2}$.

Differentiating under the integral sign, we readily obtain the $(n-1)$ partial derivatives required for finding those values of X which, consistent with statical equilibrium, will make the elastic potential a minimum. Using the same notation as before, they will be:

$$\left.\begin{aligned}
\frac{\delta U}{\delta X_4} &= \sum_{m=0}^{m=n}\left[\int_0^L \frac{T_m}{E F_m}\frac{\delta T_m}{\delta X_0}\,d\,l + \int_0^L \frac{M_{hm}}{E I_{hm}}\frac{\delta M_{hm}}{\delta X_0}\,d\,l\right] \\
&\quad + \sum_{m=1}^{m=n}\int_0^{a_m}\frac{M_{vm}}{E I_{vm}}\frac{\delta M_{vm}}{\delta X_0}\,d\,y = 0 \\
\frac{\delta U}{\delta X_1} &= \sum_{m=0}^{m=n}\left[\int_0^L \frac{T_m}{E F_m}\frac{\delta T_m}{\delta X_1}\,d\,l + \int_0^L \frac{M_{hm}}{E I_{hm}}\frac{\delta M_{hm}}{\delta X_1}\,d\,l\right] \\
&\quad + \sum_{m=1}^{m=n}\int_0^{a_m}\frac{M_{vm}}{E I_{vm}}\frac{\delta M_{vm}}{\delta X_1}\,d\,y = 0 \\
&\quad \cdot \quad \cdot \quad \cdot \quad \cdot \quad \cdot \\
\frac{\delta U}{\delta X_{n-2}} &= \sum_{m=0}^{m=n}\left[\int_0^L \frac{T_m}{E F_m}\frac{\delta T_m}{\delta X_{n-2}}\,d\,l + \int_0^L \frac{M_{hm}}{E I_{hm}}\frac{\delta M_{hm}}{\delta X_{n-2}}\,d\,l\right] \\
&\quad + \sum_{m=1}^{m=n}\int_0^{a_m}\frac{M_{vm}}{E I_{vm}}\frac{\delta M_{vm}}{\delta X_{n-2}}\,d\,y = 0
\end{aligned}\right\} \ldots (6)$$

In Equations (6) the first two terms correspond to the work done in the horizontal arches, and the last term to that in the vertical girder, so that these equations are of the form:

$$\frac{\delta U}{\delta X} = \frac{\delta U_h}{\delta X} + \frac{\delta U_v}{\delta X} = 0 \ldots\ldots\ldots\ldots (7)$$

The direct thrust, T, in a lock-gate is nearly constant throughout the length of the leaf for any given horizontal girder, and may be expressed very simply.

If $r =$ the radius of a circle which passes through the centers of bearings at the quoin and miter posts when the gate is closed; (Fig. 2) and

$P =$ the load per linear unit on the horizontal girder; then

$T = P\,r.$

For any horizontal frame, m, in a gate, therefore, we can write,

$$T_m = (P_m + X_m) r \ldots\ldots\ldots\ldots\ldots\ldots (8)$$

Also, if e_m is the mean eccentricity of the resultant in the horizontal girder, m, that is, the average distance between the line of pressures and the center of gravity of the cross-section, we can also write,

$$M_{hm} = e_m T_m = (P_m + X_m)\, e_m\, r \ldots\ldots\ldots\ldots (9)$$

The first two terms in Equations (6), which correspond to the work in the horizontal arches, therefore, may be written in the form:

$$\frac{\delta\, U_h}{\delta\, X} = r^2 \sum_{m=0}^{m=n} \frac{1}{E} \left[\frac{1}{F_m} + \frac{e_m{}^2}{I_{hm}}\right] \int_0^L \frac{(P_m + X_m)\, \delta\, (P_m + X_m)}{\delta\, X}\, d\, l \,.(10)$$

Similarly, the last term of Equations (6) and (7), corresponding to work in the vertical girder, may be easily expressed as follows:

Let M_{vm} be the bending moment in any point at any panel, m, and let the distance of this point below the panel point next above $= y$. We can then write,

$$M_{vm} = L[(h_{m-1} + y)\, X_0 + (h_{m-1} - h_1 + y)\, X_1 + \\ \ldots + (h_{m-1} - h_{m-2} + y)\, X_{m-2} + y\, X_{m-1}] \ldots (11)$$

The partial derivatives will be,

$$\frac{\delta\, M_{vm}}{\delta\, X_0} = L\,(h_{m-1} + y), \quad \frac{\delta\, M_{vm}}{\delta\, X_1} = L\,(h_{m-1} - h_1 + y)$$

$$\ldots \quad \frac{\delta\, M_{vm}}{\delta\, X_{m-1}} = L\, y.$$

That portion of the last term in Equations (6) which corresponds to the work in Panels (1) to $(n - 1)$ will then be given by the following expression:

$$\sum_{m=1}^{m=n-1} \int_0^{a_m} \frac{M_{vm}}{E\, I_{vm}} \frac{\delta\, M_{vm}}{\delta\, X_0} d\, y$$

$$= \sum_{m=1}^{m=n-1} \frac{L^2}{E\, I_{vm}} \left[\left\{ h^2_{m-1}\, a_m + 2\, h_{m-1} \frac{a_m^{\ 2}}{2} + \frac{a_m^{\ 3}}{3} \right\} X_0 \right.$$

$$+ \left\{ h_{m-1}\, (h_{m-1} - h_1)\, a_m + (2\, h_{m-1} - h_1) \frac{a_m^{\ 2}}{2} + \frac{a_m^{\ 3}}{3} \right\} X_1$$

$$\left. + \ \ . \ \ . \ \ . \ \ + \left\{ h_{m-1} \frac{a_m^{\ 2}}{2} + \frac{a_m^{\ 3}}{3} \right\} X_{m-1} \right] \ldots\ldots\ldots (12)$$

and there will be similar forms corresponding to the other variables, $X_1 \ . \ . \ . \ X_{n-2}$.

The part of the last term of Equations (6) corresponding to the work in the bottom panel must be obtained in a somewhat different way, as it is necessary to express X_n and X_{n-1} in terms of $X_0 \ . \ . \ . \ X_{n-2}$.

Let $z =$ the distance of any point in the panel from the bottom of the gate; then we can write:

$$M_{vn} = L\, z\, X_n$$

From Equations (1) and (2), if we take moments about the girder, $(n - 1)$,

$$X_n = \frac{1}{a_n} \left\{ h_{n-1}\, X_0 + (h_{n-1} - h_1)\, X_1 \right.$$

$$\left. + \ . \ \ . \ \ . \ \ (h_{n-1} - h_{n-2})\, X_{n-2} \right\}$$

hence,

$$M_{vn} + \frac{L\, z}{a_n} \left\{ h_{n-1}\, X_0 + (h_{n-1} - h_1)\, X_1 \right.$$

$$\left. + \ \ . \ \ . \ \ . \ \ (h_{n-1} - h_{n-2})\, X_{n-2} \right\} \ldots\ldots\ldots\ldots\ldots (13)$$

for which we readily derive,

$$\int_0^{a_n} \frac{M_{vn}}{E\, I_{vn}} \frac{\delta\, M_{vn}}{\delta\, X_0} = \frac{L^2\, a_n}{3\, E\, I_{vn}} [h^2_{n-1}\, X_0 + h_{n-1}\, (h_{n-1} - h_1)\, X_1$$

$$+ \ \ . \ \ . \ \ . \ \ h_{n-1}\, (h_{n-1} - h_{n-2})\, X_{n-2}] \ldots\ldots\ldots (14)$$

which represents the term corresponding to work in the bottom panel of the vertical girder. We can readily obtain similar forms when X $. \ . \ . \ X_{n-2}$ are the variables.

General Equations of Condition.—Combining Equations (10), (12), and (13), and omitting the common factor, $\frac{1}{E}$, we have the following general equations.

With X_0 as the independent variable:

$$
\begin{aligned}
\frac{\delta U}{\delta X_0} = r^2 \sum_{m=0}^{m=n} \left[\frac{1}{F_m} + \frac{e^2_m}{I_{hm}}\right] \int_0^L \frac{(P_m + X_m)\,\delta\,(P_m + X_m)}{\delta X_0}\, d\,l \\
+ L^2 \sum_{m=1}^{m=n-1} \frac{1}{I_{vm}} \left[\left\{ h^2_{m-1}\, a_m + 2\, h_{m-1} \frac{a_m^2}{2} + \frac{a_m^3}{3} \right\} X_0 \right. \\
+ \left\{ h_{m-1}\,(h_{m-1} - h_1)\, a_m + (2\, h_{m-1} - h_1) \frac{a_m^2}{2} + \frac{a_m^3}{3} \right\} X_1 \\
\left. + \; . \;\; . \;\; . \;\; + \left\{ h_{m-1} \frac{a_m^2}{2} + \frac{a_m^3}{3} \right\} X_{m-1} \right] \\
+ \frac{L^2\, a_n}{3\, I_{vn}} [h^2_{n-1}\, X_0 + h_{n-1}\,(h_{n-1} - h_1)\, X_1 \\
+ \; . \;\; . \;\; . \;\; + h_{n-1}\,(h_{n-1} - h_{n-2})\, X_{n-2}] = 0
\end{aligned}
$$

and with X_1 as the independent variable:

$$
\begin{aligned}
\frac{\delta U}{\delta X_1} = r^2 \sum_{m=0}^{m=n} \left[\frac{1}{F_m} + \frac{e_m^2}{I_{hm}}\right] \int_0^L \frac{(P_m + X_m)\,\delta\,(P_m + X_m)}{\delta X_1}\, d\,l \\
+ L^2 \sum_{m=1}^{m=n-1} \frac{1}{I_{vm}} \left[\left\{ h_{m-1}\,(h_{m-1} - h_1)\, a_m \right. \right. \\
\left. + (2\, h_{m-1} - h_1) \frac{a_m^2}{2} + \frac{a_m^3}{3} \right\} X_0 \\
+ \left\{ (h_{m-1} - h_1)^2\, a_m + 2\,(h_{m-1} - h_1) \frac{a_m^2}{2} + \frac{a_m^3}{3} \right\} X_1 + \; . \;\; . \;\; . \\
\left. + \left\{ (h_{m-1} - h_1) \frac{a_m^2}{2} + \frac{a_m^3}{3} \right\} X_{m-1} \right] \\
+ \frac{L^2\, a_n}{3\, I_{vn}} [(h_{n-1} - h_1)^2\, X_1 + \; . \;\; . \;\; . + (h_{n-1} - h_1) \\
(h_{n-1} - h_{n-2})\, X_{n-2}] = 0
\end{aligned}
\qquad \ldots (15)
$$

and equations of similar form for $X_2 \; . \; . \; . \; X_{n-2}$ as independent variables.

By making the proper substitutions and summations in Equations (15), the $(n - 2)$ simultaneous equations may be written, and from these, the values, X_1 to X_{n-1}, are readily found.

Application of Formulas to Gates of the Panama Canal.

77½-Foot Gate.—The general arrangement of the gate leaf is shown on Plate IX and the photograph, Fig. 3, which represents a somewhat lower gate before the sheathing is attached.

The principal dimensions are as follows:

Clear width of lock = 110 ft.;

Height of gate = 77 ft. 6 in. from top of coping to center of bottom girder;

Central thickness = 7 ft. 0 in.;

Shape of gate: straight-backed.

There are sixteen horizontals, the spacing of which is shown on Plate IX and Fig. 1 of Plate X, and is tabulated subsequently.

The cross-sections, moments of inertia, and eccentricities of lines of pressures, which are shown on Fig. 1, Plate X, and are also given later, are mean values for each of the horizontal arches. The moments of inertia for the vertical girder are calculated from the skin thickness of each panel and the average depth of the girder. If t_1 and t_2 are the thicknesses of the up-stream and down-stream sheathing, and d_1 and d_2 their distances from the center of gravity of the cross-sections of the girder, the moment of inertia is given by the expression:

$$I_v = L(t_1\, d_1^2 + t_2\, d_2^2).$$

Summarized, the data are as follows:

Length of leaf, $L = 787$ in.,

Radius of line of pressures, $r = 880$ in. (See Plate X, Fig. 5.)

Horizontal Arches.—Mean cross-sections:

$F_0 = F_1 = F_2 = F_3$ .... = 100 sq. in.

$F_4 = F_5$ .... = 120 " "

$F_6 = F_7 = F_{15}$ .... = 140 " "

$F_8 = F_9$ .... = 164 " "

$F_{10} = F_{11} = F_{12} = F_{13} = F_{14}$ ..... = 200 " "

Mean moments of inertia:

$I_0 = I_1 = I_2 = I_3$ .... = 120 000 in.^{4}

$I_4 = I_5$ .... = 140 000 "

$I_6 = I_7 = I_{15}$ .... = 156 000 "

$I_8 = I_9$ .... = 167 000 "

$I_{10} = I_{11} = I_{12} = I_{13} = I_{14}$ .... = 190 000 "

Mean eccentricity of line of pressure:

$e_0 = e_1 = e_2 = e_3$ .... = 26 in.
$e_4 = e_5$ .... = 21 "
$e_6 = e_7 = e_{15}$ .... = 16 "
$e_8 = e_9$ .... = 13 "
$e_{10} = e_{11} = e_{12} = e_{13} = e_{14}$ ...... = 11 "

Vertical Girder.—Moments of inertia:

$I_1 = I_2 = I_3$ .... = 1 100 000 in.^{4}
$I_4 = I_5$ .... = 1 180 000 "
$I_6 = I_7$ .... = 1 240 000 "
$I_8 = I_9 = I_{10} = I_{11}$ .... = 1 400 000 "
$I_{12} = I_{13}$ .... = 1 580 000 "
$I_{14} = I_{15}$ .... = 1 720 000 "

Vertical Panels.—

$a_1 = a_2 = a_3 = a_4$ .... = 66 in.
$a_5 = a_6 = a_7 = a_8 = a_9 = a_{10} = a_{11}$ = 60 "
$a_{12} = a_{13} = a_{14} = a_{15}$ .... = 54 "

Heights.—

$h_1 = 66$ in.	$h_6 = 384$ in.	$h_{11} = 684$ in.
$h_2 = 132$ "	$h_7 = 444$ "	$h_{12} = 738$ "
$h_3 = 198$ "	$h_8 = 504$ "	$h_{13} = 792$ "
$h_4 = 264$ "	$h_9 = 564$ "	$h_{14} = 846$ "
$h_5 = 324$ "	$h_{10} = 624$ "	$h_{15} = 900$ "

Water Pressure.—The water is assumed to extend to the top of the coping on the up-stream side of the gate, with a pool 9 ft. 6 in. deep below. Fig. 1, Plate X, shows the total water pressure and Fig. 2, Column *C*, of that plate, the load per linear foot on each horizontal arch.

Reduced to the linear inch of girder, these values are:

$p_0 = 73.34$ lb.	$p_5 = 768.25$ lb.	$p_{10} = 1\,419.35$ lb.
$p_1 = 229.15$ "	$p_6 = 898.45$ "	$p_{11} = 1\,467.9$ "
$p_2 = 386.67$ "	$p_7 = 1\,028.65$ "	$p_{12} = 1\,500$ "
$p_3 = 544.13$ "	$p_8 = 1\,158.9$ "	$p_{13} = 1\,580.7$ "
$p_4 = 665.38$ "	$p_9 = 1\,289.$ "	$p_{14} = 1\,593.75$ "
		$p_{15} = 796.87$ "

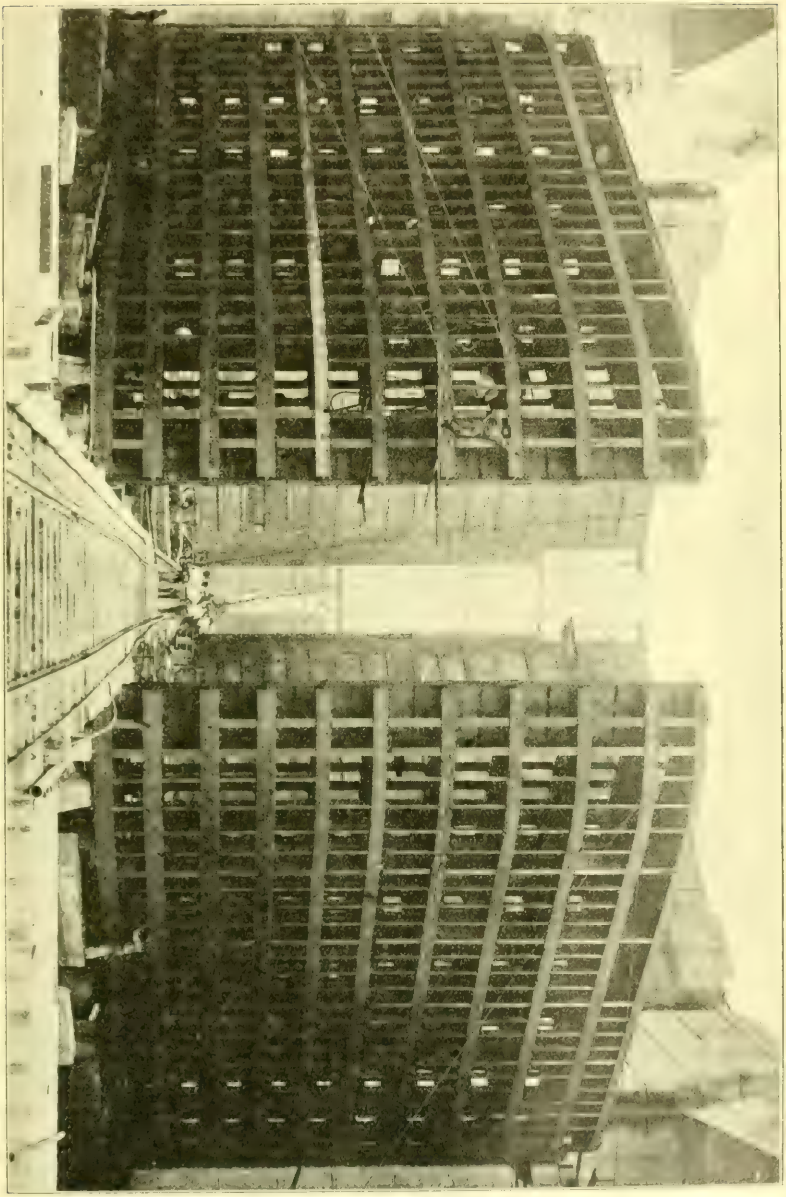
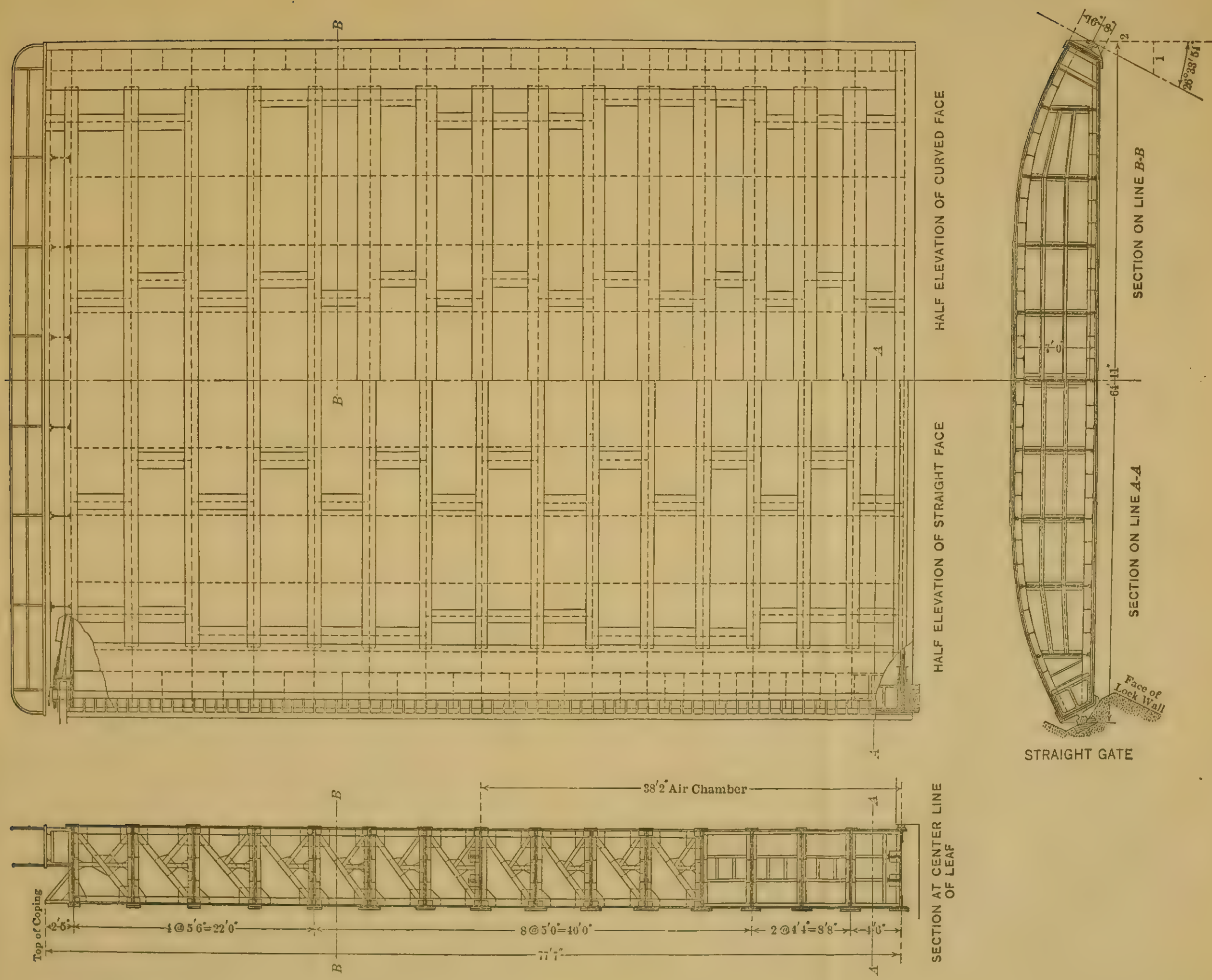


FIG. 8.—LEAVES 72 AND 73 OF PEDRO MIGUEL LOCK, PANAMA CANAL. LEAVES 68 AND 69 IN THE BACKGROUND.

PLATE IX.
PAPERS, AM. SOC. C. E.
AUGUST, 1917.
GOLDMARK ON
PANAMA LOCK GATES.



The conditions of static equilibrium, as applied to the vertical girder, become:

$$X_{15} = -(X_0 + X_1 + X_2 + X_3 + X_4 + X_5 + X_6 + X_7 + X_8 + X_9 + X_{10} + X_{11} + X_{12} + X_{13} + X_{14}) \ldots\ldots\ldots\ldots (16)$$

and,

$$X_{14} = -\frac{1}{54}(900\,X_0 + 834\,X_1 + 768\,X_2 + 702\,X_3 + 636\,X_4 + 576\,X_5 + 516\,X_6 + 456\,X_7 + 396\,X_8 + 336\,X_9 + 276\,X_{10} + 216\,X_{11} + 162\,X_{12} + 108\,X_{13})$$

whence,

$$X_{14} = -(16.66\,X_0 + 15.44\,X_1 + 14.22\,X_2 + 13\,X_3 + 11.78\,X_4 + 10.67\,X_5 + 9.56\,X_6 + 8.44\,X_7 + 7.33\,X_8 + 6.22\,X_9 + 5.11\,X_{10} + 4\,X_{11} + 3\,X_{12} + 2\,X_{13}) \ldots\ldots\ldots\ldots (17)$$

and,

$$X_{15} = +15.66\,X_0 + 14.44\,X_1 + 13.22\,X_2 + 12\,X_3 + 10.78\,X_4 + 9.67\,X_5 + 8.56\,X_6 + 7.44\,X_7 + 6.33\,X_8 + 5.22\,X_9 + 4.11\,X_{10} + 3\,X_{11} + 2\,X_{12} + X_{13} \ldots\ldots\ldots\ldots (18)$$

In order to introduce the static conditions of equilibrium, the values of X_{14} and X_{15} in terms of $X_0 \ldots X_{13}$, as given in Equations (17) and (18), will be used as shown below.

The resulting simultaneous equations of condition, corresponding to $\frac{\delta U}{\delta X_0} = 0$; $\frac{\delta U}{\delta X_1} = 0$, etc., will then be reduced to fourteen, with $X_0 \ldots X_{13}$ as variables.

Work of Horizontal Arches.

The first terms of Equations (15) correspond to the total work of the horizontals.

Each arch must be taken up separately, and the values of the expression, $r^2\left(\frac{1}{F_m} + \frac{e^2{}_m}{I_{hm}}\right)\int_0^L (P_m + X_m)\frac{\delta(P_m + X_m)}{\delta X}\,d\,l$, obtained, taking $X \ldots X_{13}$, successively as variables. It may be convenient to represent this expression generally by the symbol H. Its values for the separate arches and variables will be the following:

Arch 0:

X_0 as variable:

$$H = 12\,106\int_0^L (P_0 + X_0)\frac{\delta(P_0 + X_0)\,d\,l}{\delta X_0} = 12\,106\,(P_0 + X_0)\,L$$

$$= 12\,106\,(78.34 + X_0)\,L = (12\,106\,X_0 + 948\,384)\,L$$

X_1 as variable:

$$H = 12\,106 \int_0^L (P_0 + X_0) \frac{\delta\,(P_0 + X_0)\,d\,l}{\delta\,X_1} = 0$$

and, similarly, we should find that $H = 0$ when $X_2 \ldots X_{13}$ are the variables.

Arch 1:

X_0 as variable:

$$H = 12\,106 \int_0^L (P_1 + X_1) \frac{\delta\,(P_1 + X_1)\,d\,l}{\delta\,X_0} = 0$$

X_1 as variable:

$$H = 12\,106 \int_0^L (P_1 + X_1) \frac{\delta\,(P_1 + X_1)\,d\,l}{\delta\,X_1} = 12\,106\,(229.15 + X_1)\,L$$

$$= (12\,106\,X_1 + 2\,774\,090)\,L.$$

For $X_2 \ldots X_{13}$, $H = 0$.

In like manner, for the arches, 3, 4 . . . 13, H will $= 0$ for every variable but one in the case of each arch. There will be a significant value, however, in the case of each arch for the one variable which has the same number as the arch in question.

These values of $H = r^2 \left(\frac{1}{F} + \frac{e^2}{I_h}\right) \int_0^L (P + X) \frac{(P + X)}{X}\,d\,l$ will be the following:

Arch 0.—X_0	as independent variable;	$H = 12\,106\,X_0 + 948\,384$
Arch 1.—X_1	" " "	$H = 12\,106\,X_1 + 2\,774\,090$
Arch 2.—X_2	" " "	$H = 12\,106\,X_2 + 4\,681\,027$
Arch 3.—X_3	" " "	$H = 12\,106\,X_3 + 6\,587\,238$
Arch 4.—X_4	" " "	$H = 8\,892\,X_4 + 5\,916\,559$
Arch 5.—X_5	" " "	$H = 8\,892\,X_5 + 6\,831\,279$
Arch 6.—X_6	" " "	$H = 6\,802\,X_6 + 6\,111\,257$
Arch 7.—X_7	" " "	$H = 6\,802\,X_7 + 6\,996\,877$
Arch 8.—X_8	" " "	$H = 5\,477\,X_8 + 6\,347\,295$
Arch 9.—X_9	" " "	$H = 5\,477\,X_9 + 7\,059\,853$
Arch 10.—X_{10}	" " "	$H = 4\,365\,X_{10} + 6\,195\,463$
Arch 11.—X_{11}	" " "	$H = 4\,365\,X_{11} + 6\,407\,383$
Arch 12.—X_{12}	" " "	$H = 4\,365\,X_{12} + 6\,547\,500$
Arch 13.—X_{13}	" " "	$H = 4\,365\,X_{13} + 6\,899\,755$

Arch 14.—For this arch, the general form of H will be

$$H = 4\,365 \int_0^L (P_{14} + X_{14}) \frac{\delta\,(P_{14} + X_{14})}{\delta\,X}\,dl$$

$$= 4\,365 \int_0^L (1\,593.75 + X_{14}) \frac{\delta\,(1\,593.75 + X_{14})}{\delta\,X}\,dl$$

77'6" GATE FOR 110-FOOT LOCK

Horizontal Girders

Moment of inertia, in (inches)⁴	Area of cross-section, in square inches.	Eccentricity of load, in inches.
120 000	100	26
120 000	100	26
120 000	100	26
120 000	100	26
140 000	120	21
140 000	120	21
156 000	140	16
156 000	140	16
167 000	164	13
167 000	164	13
190 000	200	11
190 000	200	11
190 000	200	11
190 000	200	11
190 000	200	11
156 000	140	16

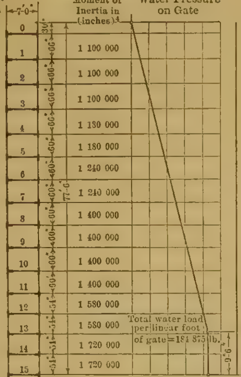


FIG. 1.

Loads on Horizontal Girders

A Sill immovable, Vertical girder acting. Plotted thus
B " removed, " " " "
C " " " " removed. " "

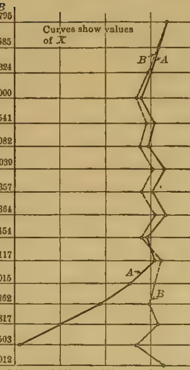
Girder	*A*	*C*	*B*
0	2 815	940	2 795
1	3 725	2 750	3 585
2	4 643	4 640	4 324
3	5 476	6 530	5 000
4	8 317	7 985	7 541
5	8 946	9 220	8 082
6	12 271	10 780	11 039
7	12 524	12 345	11 357
8	15 541	13 910	14 364
9	15 005	15 470	14 454
10	17 480	17 035	18 117
11	15 230	17 615	18 015
12	12 405	18 000	17 862
13	8 920	18 970	19 817
14	4 635	19 125	17 503
15	36 882	9 560	11 012
		184 875 Total	

Note:-This load is carried entirely by the sill.

FIG. 2.

Reactions of Vertical Girder on Horizontal Girders

A Sill immovable, Plotted thus
B " removed, " "



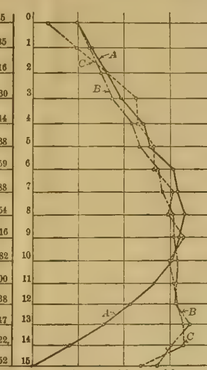
Girder	*A*	*B*
0	+ 1 875	+1 855
1	+ 975	+ 835
2	+ 3	− 316
3	− 1 054	−1 530
4	+ 332	− 444
5	− 274	− 1 138
6	+ 1 491	+ 259
7	+ 179	− 988
8	+ 1 631	+ 454
9	− 465	− 1 016
10	+ 445	+1 082
11	− 2 385	+ 400
12	− 5 535	− 138
13	− 10 050	+ 847
14	− 14 490	−1 622
15	+27 322	+ 1 452

Note:-This reaction is carried by the sill.

FIG. 3.

Deflection of Miter Post under Water Pressure

A Sill immovable, Vertical girder acting. Plotted thus
B " removed, " " " "
C " " " " removed. " "



Girder	*A*	*C*	*B*
0	0.0980	0.0332	0.0973
1	0.1296	0.0958	0.1249
2	0.1616	0.1617	0.1505
3	0.1906	0.2274	0.1741
4	0.2411	0.2318	0.2186
5	0.2591	0.2673	0.2346
6	0.3050	0.2682	0.2745
7	0.3153	0.3068	0.2825
8	0.3296	0.2950	0.3050
9	0.3184	0.3283	0.3070
10	0.3040	0.2964	0.3158
11	0.2650	0.3068	0.3138
12	0.2167	0.3134	0.3110
13	0.1551	0.3303	0.3451
14	0.0806	0.3326	0.3060
15	0.	0.2376	0.2740

FIG. 4.

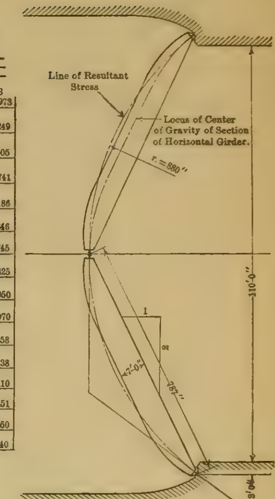


FIG. 5.

49'6" GATE FOR 110-FOOT LOCK

Horizontal Girders

Moment of inertia, in (inches)⁴	Area of cross-section, in square inches.	Eccentricity of load, in inches.
120 000	100	26
120 000	100	26
120 000	100	26
120 000	100	26
140 000	120	21
140 000	120	21
156 000	140	16
156 000	140	16
167 000	164	13
140 000	120	21

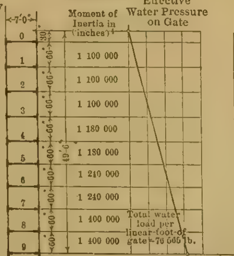
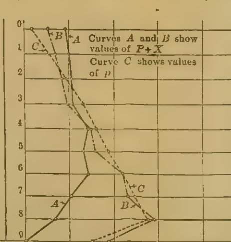


FIG. 6.

Loads on Horizontal Girders

A Sill immovable, Vertical girder acting. Plotted thus
B " removed, " " " "
C " " " " removed. " "



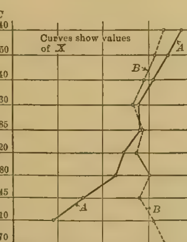
Girder	*A*	*B*	*C*
0	4 672	2 724	940
1	4 963	3 485	2 750
2	5 269	4 202	4 640
3	5 374	4 851	6 530
4	7 096	7 342	7 985
5	6 519	7 897	9 220
6	7 163	10 892	10 780
7	5 224	11 356	12 345
8	3 447	14 578	13 910
9	26 857	9 242	7 470
		Total 76 570	

Note:-This load is carried entirely by the sill.

FIG. 7.

Reactions of Vertical Girder on Horizontal Girders

A Sill immovable, Plotted thus
B " removed, " "



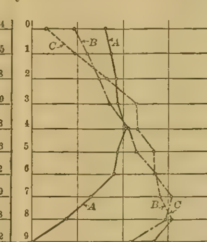
Girder	*A*	*B*
0	+ 3 732	+1 784
1	+2 213	+ 735
2	+ 629	− 438
3	− 1 156	−1 079
4	− 889	− 643
5	− 2 701	−1 323
6	− 3 617	+ 112
7	− 7 121	− 989
8	−10 403	+ 668
9	+19 387	+1 772

Note:- This reaction is carried by the sill.

FIG. 8.

Deflection of Miter Post under Water Pressure

A Sill immovable, Vertical girder acting. Plotted thus
B " removed, " " " "
C " " " " removed. " "



Girder	*A*	*B*	*C*
0	0.1624	0.0948	0.0327
1	0.1726	0.1212	0.0957
2	0.1832	0.1462	0.1614
3	0.1870	0.1688	0.2270
4	0.2050	0.2128	0.2313
5	0.1889	0.2288	0.2672
6	0.1780	0.2707	0.2680
7	0.1297	0.2822	0.3068
8	0.0732	0.3000	0.2952
9	0.	0.2680	0.2165

FIG. 9.

Substituting for X_{14} its value from Equation (17), we have

$$H = 4\,365 \int_0^L (1\,593.75 - 16.66\,X_0 \quad . \quad . \quad . \quad - 2\,X_{13})$$

$$\left[\frac{1}{\delta\,X}\,\delta\,(1\,593.75 - 16.66\,X_0 \quad . \quad . \quad . \quad - 2\,X_{13})\right] d\,l.$$

For the different independent variables, we have then total values for H:

X_0 as variable:

$$H = 4\,365\ (-\ 16.66)\ (1\,593.75 - 16.66\,X_0\ .\ .\ .\ - 2\,X_{13})\ L =$$
$$(-\ 115\,898\,930 + 1\,211\,500\,X_0 + 1\,122\,800\,X_1 + 1\,034\,000\,X_2$$
$$+\ 945\,370\,X_3 + 856\,650\,X_4 + 775\,900\,X_5 + 694\,920\,X_6$$
$$+\ 614\,100\,X_7 + 533\,330\,X_8 + 452\,500\,X_9 + 371\,700\,X_{10}$$
$$+\ 290\,883\,X_{11} + 218\,163\,X_{12} + 145\,400\,X_{13})\ L$$

X_1 as variable:

$$H = 4\,365\ (-\ 15.44)\ (1\,593.75 - 16.66\,X_0\ .\ .\ .\ - 2\,X_{13})\ L =$$
$$(-\ 107\,411\,700 + 1\,122\,800\,X_0 + 1\,040\,590\,X_1 + 958\,400\,X_2$$
$$+\ 876\,140\,X_3 + 793\,920\,X_4 + 719\,110\,X_5 + 644\,030\,X_6$$
$$+\ 569\,090\,X_7 + 494\,280\,X_8 + 419\,330\,X_9 + 344\,460\,X_{10}$$
$$+\ 269\,580\,X_{11} + 202\,190\,X_{12} + 134\,790\,X_{13})\ L$$

and similar expressions when the differentiation is made with reference to the other independent variables, $X_2\ .\ .\ .\ X_{13}$.

These values are not copied out in full, as the work is very voluminous. Similarly, the method is only indicated in the case of Arch 15.

Arch 15.—For this the general expression for H is:

$$H = 6\,802 \int_0^L (P_{15} + X_{15})\,\frac{\delta\,(P_{15} + X_{15})}{\delta\,X}\,d\,l$$

$$= 6\,802 \int_0^L (796.87 + X_{15})\,\frac{\delta\,(796.87 + X_{15})}{\delta\,X}\,d\,l.$$

If we substitute for X_{15} the value from Equation (18), we have

$$H = 6\,802 \int_0^L (796.87 + 15.66\,X_0 + 14.44\,X_1 + 13.22\,X_2$$
$$+\ 12\,X_3 + 10.78\,X_4 + 9.67\,X_5 + 8.56\,X_6 + 7.44\,X_7$$
$$+\ 6.33\,X_8 + 5.22\,X_9 + 4.11\,X_{10} + 3\,X_{11} + 2\,X_{12} + X_{13})$$
$$\times\ \frac{\delta\,(796.87 + 15.66\,X_0\ \ .\ \ .\ \ .\ \ X_{13})}{\delta\,X}\,d\,l$$

which, for the different independent variables, becomes:

X_0 as variable:

$$H = 6\,802\ (15.66)\ (796.87 + 15.66\ X_0\ .\ .\ .\ X_{13})\ L = (84\,918\,190 + 1\,669\,518\ X_0 + 1\,539\,272\ X_1 + 1\,409\,026\ X_2 + 1\,278\,780\ X_3 + 1\,148\,534\ X_4 + 1\,030\,128\ X_5 + 911\,723\ X_6 + 793\,317\ X_7 + 674\,912\ X_8 + 556\,506\ X_9 + 438\,101\ X_{10} + 319\,695\ X_{11} + 213\,130\ X_{12} + 106\,565\ X_{13})\ L$$

and similar expressions when the differentiation is made with reference to the other independent variables, $X_1\ .\ .\ .\ X_{13}$.

Vertical Girder.

The second term of Equations (15) gives that part of the final equations due to the work in the vertical girder. If this term, for convenience, is called V, its value for the several panels and variables will be the following:

Panel 1.—X_0 as variable:

$$V = 0.0871\ L^2\ X_0$$

and the values for the other variables vanish.

Panel 2.—X_0 as variable:

$$V = + 0.6097\ L^2\ X_0 + 0.2175\ L^2\ X_1$$

X_1 as variable:

$$V = 0.2175\ L^2\ X_0 + 0.0871\ L^2\ X_1$$

the values for the other variables being equal to 0.

Panels 3-15.—The corresponding expressions for the remaining panels of the vertical girder will be of the same general form as those deduced for the first two panels.

In obtaining the value of V for the bottom panel (15) the variable, X, was, of course, expressed in terms of $X_0\ .\ .\ .\ X_{13}$.

Method of Obtaining Final Equations.

The final equations are obtained from the values for H and for V above by adding all the terms which correspond to the same independent variable, $X_0, X_1\ .\ .\ .\ X_{13}$, and equating them to zero.

In the case of no contact at the sill, the values of H for the bottom girder (15) must be included, as this girder then carries the full load which comes on it.

For perfect contact, the bottom girder does no work as an arch, and hence the H's for this girder must be omitted.

If the contact is such that the bottom girder does some work in arch action, a portion of its H's may be counted. In other respects, the method of calculation is independent of the degree of sill contact.

It should be noted that L (the length of the leaf, 787 in.) enters all values of H in the first power and those of V in the second. By dividing the final equations by L, it will be eliminated from the H terms, but will remain in the first power in those corresponding to V.

From their method of derivation, the resulting final equations should be symmetrical as to coefficients, and, in the equations for the solution of the problem, an average has been used where two coefficients which should be identical have differed by a slight amount. There have been no large discrepancies, proving to a considerable extent the accuracy of the work.

Only five significant figures were retained.

Final Equations of Conditions.—The final equations, as thus obtained, are the following:

(Case A).—No Contact at Sill.

X_0 as variable:

$$3\,007\,500\ X_0 + 2\,762\,700\ X_1 + 2\,530\,200\ X_2 + 2\,298\,200\ X_3 + 2\,066\,800\ X_4 + 1\,856\,900\ X_5 + 1\,647\,600\ X_6 + 1\,439\,300\ X_7 + 1\,232\,100\ X_8 + 1\,025\,700\ X_9 + 820\,580\ X_{10} + 616\,660\ X_{11} + 434\,260\ X_{12} + 252\,900\ X_{13} - 30\,032\,300 = 0.$$

X_1 as variable:

$$2\,762\,700\ X_0 + 2\,561\,000\ X_1 + 2\,335\,100\ X_2 + 2\,121\,400\ X_3 + 1\,908\,200\ X_4 + 1\,714\,800\ X_5 + 1\,521\,700\ X_6 + 1\,329\,500\ X_7 + 1\,138\,200\ X_8 + 947\,700\ X_9 + 758\,250\ X_{10} + 569\,920\ X_{11} + 401\,420\ X_{12} + 233\,890\ X_{13} - 26\,344\,200 = 0.$$

X_2 as variable:

$$2\,530\,200\ X_0 + 2\,335\,100\ X_1 + 2\,151\,900\ X_2 + 1\,944\,600\ X_3 + 1\,749\,600\ X_4 + 1\,572\,600\ X_5 + 1\,395\,800\ X_6 + 1\,219\,700\ X_7 + 1\,044\,400\ X_9 + 869\,700\ X_9 + 695\,950\ X_{10} + 523\,170\ X_{11} + 368\,580\ X_{12} + 214\,860\ X_{13} - 22\,574\,900 = 0.$$

X_3 as variable:

$$2\,298\,200\ X_0 + 2\,121\,400\ X_1 + 1\,944\,600\ X_2 + 1\,780\,000\ X_3 + 1\,591\,000\ X_4 + 1\,430\,400\ X_5 + 1\,269\,900\ X_6 + 1\,109\,900\ X_7 + 950\,550\ X_8 + 791\,700\ X_9 + 633\,630\ X_{10} + 476\,420\ X_{11} + 335\,730\ X_{12} + 195\,820\ X_{13} - 18\,806\,300 = 0.$$

X_4 as variable:

$$2\,066\,800\ X_0 + 1\,908\,200\ X_1 + 1\,749\,600\ X_2 + 1\,591\,000\ X_3 + 1\,441\,400\ X_4 + 1\,288\,300\ X_5 + 1\,144\,000\ X_6 + 1\,000\,100\ X_7 + 856\,700\ X_8 + 713\,680\ X_9 + 571\,330\ X_{10} + 429\,690\ X_{11} + 302\,890\ X_{12} + 176\,790\ X_{13} - 17\,614\,700 = 0.$$

X_5 as variable:

$$1\,856\,900\ X_0 + 1\,714\,800\ X_1 + 1\,572\,600\ X_2 + 1\,430\,400\ X_3 + 1\,288\,300\ X_4 + 1\,167\,800\ X_5 + 1\,029\,500\ X_6 + 900\,250\ X_7 + 771\,370\ X_8 + 642\,750\ X_9 + 514\,660\ X_{10} + 387\,180\ X_{11} + 273\,010\ X_{12} + 159\,470\ X_{13} - 15\,000\,500 = 0.$$

X_6 as variable:

$$1\,647\,600\ X_0 + 1\,521\,700\ X_1 + 1\,395\,800\ X_2 + 1\,269\,900\ X_3 + 1\,144\,000\ X_4 + 1\,029\,500\ X_5 + 921\,620\ X_6 + 800\,250\ X_7 + 685\,900\ X_8 + 571\,700\ X_9 + 457\,910\ X_{10} + 344\,600\ X_{11} + 243\,110\ X_{12} + 142\,120\ X_{13} - 13\,993\,300 = 0.$$

X_7 as variable:

$$1\,439\,300\ X_0 + 1\,329\,500\ X_1 + 1\,219\,700\ X_2 + 1\,109\,900\ X_3 + 1\,000\,100\ X_4 + 900\,250\ X_5 + 800\,250\ X_6 + 707\,130\ X_7 + 600\,490\ X_8 + 500\,700\ X_9 + 401\,200\ X_{10} + 302\,050\ X_{11} + 213\,200\ X_{12} + 124\,790\ X_{13} - 11\,394\,500 = 0.$$

X_8 as variable:

$$1\,232\,100\ X_0 + 1\,138\,200\ X_1 + 1\,044\,400\ X_2 + 950\,550\ X_3 + 856\,700\ X_4 + 771\,370\ X_5 + 685\,900\ X_6 + 600\,490\ X_7 + 520\,620\ X_8 + 429\,760\ X_9 + 344\,530\ X_{10} + 259\,540\ X_{11} + 183\,340\ X_{12} + 107\,470\ X_{13} - 10\,344\,600 = 0.$$

X_9 as variable:

$$1\,025\,700\ X_0 + 947\,700\ X_1 + 869\,700\ X_2 + 791\,700\ X_3 + 713\,680\ X_4 + 642\,750\ X_5 + 571\,700\ X_6 + 500\,700\ X_7 + 429\,760\ X_8 + 364\,240\ X_9 + 287\,820\ X_{10} + 217\,000\ X_{11} + 153\,440\ X_{12} + 90\,138\ X_{13} - 7\,918\,740 = 0.$$

X_{10} as variable:

$$820\,580\ X_0 + 758\,250\ X_1 + 695\,950\ X_2 + 633\,630\ X_3 + 571\,330\ X_4 + 514\,660\ X_5 + 457\,910\ X_6 + 401\,200\ X_7 + 344\,530\ X_8 + 287\,820\ X_9 + 235\,500\ X_{10} + 174\,470\ X_{11} + 123\,560\ X_{12} + 72\,819\ X_{13} - 7\,076\,810 = 0.$$

X_{11} as variable:

$$616\,660\ X_0 + 569\,920\ X_1 + 523\,170\ X_2 + 476\,420\ X_3 + 429\,690\ X_4 + 387\,180\ X_5 + 344\,600\ X_6 + 302\,050\ X_7 + 259\,540\ X_8 + 217\,000\ X_9 + 174\,470\ X_{10} + 136\,310\ X_{11} + 93\,679\ X_{12} + 55\,495\ X_{13} - 5\,158\,560 = 0.$$

X_{12} as variable:

$$434\,260\ X_0 + 401\,420\ X_1 + 368\,580\ X_2 + 335\,730\ X_3 + 302\,890\ X_4 + 273\,010\ X_5 + 243\,110\ X_6 + 213\,200\ X_7 + 183\,340\ X_8 + 153\,440\ X_9 + 123\,560\ X_{10} + 93\,679\ X_{11} + 71\,149\ X_{12} + 39\,902\ X_{13} - 3\,482\,040 = 0.$$

X_{13} as variable:

$$252\,900\ X_0 + 233\,890\ X_1 + 214\,860\ X_2 + 195\,820\ X_3 + 176\,790\ X_4 + 159\,470\ X_5 + 142\,120\ X_6 + 124\,790\ X_7 + 107\,470\ X_8 + 90\,138\ X_9 + 72\,819\ X_{10} + 55\,495\ X_{11} + 39\,902\ X_{12} + 28\,675\ X_{13} - 1\,593\,370 = 0.$$

The method used for solving these equations is given in the Appendix. The values obtained were the following:

$X_0 = + 154.60$	$X_5 = - 94.82$	$X_{10} = + 90.18$
$X_1 = + 69.59$	$X_6 = + 21.62$	$X_{11} = + 33.31$
$X_2 = - 26.30$	$X_7 = - 82.31$	$X_{12} = - 11.51$
$X_3 = - 127.50$	$X_8 = + 37.81$	$X_{13} = + 70.62$
$X_4 = - 36.99$	$X_9 = - 84.21$	

and, from Equations (21) and (22),

$$X_{14} = - 135.1$$
$$X_{15} = + 121.0$$

These values are in pounds per linear inch of horizontal arch. In Fig. 3 (*B*), of Plate X, they are given per linear foot. Here, as in all other cases when X is positive, the vertical girder presses against the horizontals in a down-stream direction.

(Case B).—Perfect Contact at Sill.

X_0 as variable:

$$1\,338\,000\ X_0 + 1\,223\,000\ X_1 + 1\,121\,000\ X_2 + 1\,019\,000\ X_3 + 918\,300\ X_4 + 826\,800\ X_5 + 735\,900\ X_6 + 646\,000\ X_7 + 557\,200\ X_8 + 469\,200\ X_9 + 382\,500\ X_{10} + 297\,000\ X_{11} + 221\,100\ X_{12} + 146\,300\ X_{13} - 115\,000\,000 = 0.$$

X_1 as variable:

$$1\,223\,000\ X_0 + 1\,142\,000\ X_1 + 1\,036\,000\ X_2 + 942\,400\ X_3 + 849\,300\ X_4 + 765\,000\ X_5 + 681\,100\ X_6 + 598\,100\ X_7 + 516\,000\ X_8 + 434\,600\ X_9 + 354\,300\ X_{10} + 275\,200\ X_{11} + 204\,900\ X_{12} + 135\,600\ X_{13} - 104\,600\,000 = 0.$$

X_2 as variable:

$$1\,121\,000\ X_0 + 1\,036\,000\ X_1 + 962\,800\ X_2 + 865\,400\ X_3 + 780\,300\ X_4 + 703\,200\ X_5 + 626\,300\ X_6 + 550\,200\ X_7 + 474\,800\ X_8 + 400\,000\ X_9 + 326\,200\ X_{10} + 253\,400\ X_{11} + 188\,700\ X_{12} + 124\,900\ X_{13} - 94\,240\,000 = 0.$$

X_3 as variable:

$$1\,019\,000\ X_0 + 942\,400\ X_1 + 865\,400\ X_2 + 800\,500\ X_3 + 711\,300\ X_4 + 641\,400\ X_5 + 571\,600\ X_6 + 502\,300\ X_7 + 433\,600\ X_8 + 365\,400\ X_9 + 298\,100\ X_{10} + 231\,500\ X_{11} + 172\,500\ X_{12} + 114\,200\ X_{13} - 83\,850\,000 = 0.$$

X_4 as variable:

$$918\,300\ X_0 + 849\,300\ X_1 + 780\,300\ X_2 + 711\,300\ X_3 + 651\,200\ X_4 + 579\,600\ X_5 + 516\,800\ X_6 + 454\,300\ X_7 + 392\,400\ X_8 + 330\,800\ X_9 + 269\,900\ X_{10} + 209\,800\ X_{11} + 156\,300\ X_{12} + 103\,500\ X_{13} - 76\,030\,000 = 0.$$

X_5 as variable:

$$826\,800\ X_0 + 765\,000\ X_1 + 703\,700\ X_2 + 641\,400\ X_3 + 579\,600\ X_4 + 532\,200\ X_5 + 466\,900\ X_6 + 410\,800\ X_7 + 354\,900\ X_8 + 299\,400\ X_9 + 244\,300\ X_{10} + 189\,900\ X_{11} + 141\,500\ X_{12} + 93\,720\ X_{13} - 67\,400\,000 = 0.$$

X_6 as variable:

$$735\,900\ X_0 + 681\,100\ X_1 + 626\,300\ X_2 + 571\,600\ X_3 + 516\,800\ X_4 + 466\,900\ X_5 + 423\,700\ X_6 + 367\,000\ X_7 + 317\,300\ X_8 + 267\,800\ X_9 + 218\,700\ X_{10} + 170\,000\ X_{11} + 126\,700\ X_{12} + 83\,930\ X_{13} - 60\,370\,000 = 0.$$

X_7 as variable:

$$646\,000\ X_0 + 598\,100\ X_1 + 550\,200\ X_2 + 502\,300\ X_3 + 454\,300\ X_4 + 410\,800\ X_5 + 367\,000\ X_6 + 330\,200\ X_7 + 279\,800\ X_8 + 236\,300\ X_9 + 193\,000\ X_{10} + 150\,100\ X_{11} + 111\,900\ X_{12} + 74\,150\ X_{13} - 51\,750\,000 = 0.$$

X_8 as variable:

$$557\,200\ X_0 + 516\,000\ X_1 + 474\,800\ X_2 + 433\,600\ X_3 + 392\,400\ X_4 + 354\,900\ X_5 + 317\,300\ X_6 + 279\,800\ X_7 + 247\,800\ X_8 + 204\,800\ X_9 + 167\,400\ X_{10} + 130\,300\ X_{11} + 97\,180\ X_{12} + 64\,400\ X_{13} - 44\,670\,000 = 0.$$

X_9 as variable:

$$469\,200\ X_0 + 434\,600\ X_1 + 400\,000\ X_2 + 365\,400\ X_3 + 330\,800\ X_4 + 299\,400\ X_5 + 267\,800\ X_6 + 236\,300\ X_7 + 204\,800\ X_8 + 178\,700\ X_9 + 141\,800\ X_{10} + 110\,400\ X_{11} + 82\,400\ X_{12} + 54\,620\ X_{13} - 36\,220\,000 = 0.$$

X_{10} as variable:

$$382\,500\ X_0 + 354\,300\ X_1 + 326\,200\ X_2 + 298\,100\ X_3 + 269\,900\ X_4 + 244\,300\ X_5 + 218\,700\ X_6 + 193\,000\ X_7 + 167\,400\ X_8 + 141\,800\ X_9 + 120\,500\ X_{10} + 90\,580\ X_{11} + 67\,630\ X_{12} + 44\,850\ X_{13} - 29\,360\,000 = 0.$$

X_{11} as variable:

$$297\,000\ X_0 + 275\,200\ X_1 + 253\,400\ X_2 + 231\,500\ X_3 + 209\,800\ X_4 + 189\,900\ X_5 + 170\,000\ X_6 + 150\,100\ X_7 + 130\,300\ X_8 + 110\,400\ X_9 + 90\,580\ X_{10} + 75\,090\ X_{11} + 52\,870\ X_{12} + 35\,090\ X_{13} - 21\,420\,000 = 0.$$

X_{12} as variable:

$$221\,100\ X_0 + 204\,900\ X_1 + 188\,700\ X_2 + 172\,500\ X_3 + 156\,300\ X_4 + 141\,500\ X_5 + 126\,700\ X_6 + 111\,900\ X_7 + 97\,180\ X_8 + 82\,400\ X_9 + 67\,630\ X_{10} + 52\,870\ X_{11} + 43\,940\ X_{12} + 26\,300\ X_{13} - 14\,320\,000 = 0.$$

X_{13} as variable:

$$146\,300\ X_0 + 135\,600\ X_1 + 124\,900\ X_2 + 114\,200\ X_3 + 103\,500\ X_4 + 93\,720\ X_5 + 83\,930\ X_6 + 74\,150\ X_7 + 64\,400\ X_8 + 54\,620\ X_9 + 44\,850\ X_{10} + 35\,090\ X_{11} + 26\,300\ X_{12} + 21\,870\ X_{13} - 7\,014\,000 = 0.$$

The values of the variables in these equations are:

$X_0 = +\ 156.10$	$X_5 = -\ \ 22.82$	$X_{10} = +\ \ 37.12$
$X_1 = +\ \ 81.23$	$X_6 = +\ 124.1$	$X_{11} = -\ 198.5$
$X_2 = +\ \ \ 0.2828$	$X_7 = +\ \ 14.93$	$X_{12} = -\ 460.7$
$X_3 = -\ \ 87.82$	$X_8 = +\ 135.9$	$X_{13} = -\ 837.5$
$X_4 = +\ \ 27.67$	$X_9 = -\ \ 38.78$	

and, substituting in Equations (21) and (22),

$$X_{14} = -1\,208.0$$
$$X_{15} = +2\,276.0$$

all these values being pounds per linear inch. In Fig. 3 (A), of Plate X, these values are given in pounds per linear foot.

49½-Foot Gate.—This gate consists of the nine panels of the 77½-ft. gate which are nearest the top, but the bottom girder, (9), is somewhat modified, so that $F_9 = 120$ sq. in.; $I_9 = 140\,000$ in.^{4}; $e_9 = 21$ in.; and $P_9 = 622.8$ lb. per lin. in.

The static equations for equilibrium give:

$$X_8 = -(9.4\,X_0 + 8.3\,X_1 + 7.2\,X_2 + 6.1\,X_3 + 5\,X_4 + 4\,X_5 + 3\,X_6 + 2\,X_7)$$
$$X_9 = +(5.4\,X_0 + 7.3\,X_1 + 6.2\,X_2 + 5.1\,X_3 + 4\,X_4 + 3\,X_5 + 2\,X_6 + X_7)$$

The equations of conditions become:

(*Case A*).—*No Contact at Sill.*

$$1\,152\,900\,X_0 + 996\,270\,X_1 + 851\,980\,X_2 + 708\,100\,X_3 + 564\,850\,X_4 + 435\,260\,X_5 + 306\,440\,X_6 + 178\,480\,X_7 - 12\,197\,513 = 0.$$
$$996\,270\,X_0 + 882\,620\,X_1 + 744\,830\,X_2 + 619\,360\,X_3 + 494\,280\,X_4 + 381\,050\,X_5 + 268\,440\,X_6 + 156\,550\,X_7 - 9\,481\,511 = 0.$$
$$851\,980\,X_0 + 744\,830\,X_1 + 649\,790\,X_2 + 530\,590\,X_3 + 423\,710\,X_4 + 326\,850\,X_5 + 230\,440\,X_6 + 134\,610\,X_7 - 6\,684\,281 = 0.$$
$$708\,100\,X_0 + 619\,360\,X_1 + 530\,590\,X_2 + 453\,960\,X_3 + 353\,120\,X_4 + 272\,640\,X_5 + 192\,440\,X_6 + 112\,660\,X_7 - 3\,887\,777 = 0.$$
$$564\,850\,X_0 + 494\,280\,X_1 + 423\,710\,X_2 + 353\,120\,X_3 + 291\,460\,X_4 + 218\,430\,X_5 + 154\,440\,X_6 + 90\,720\,X_7 - 3\,668\,164 = 0.$$
$$435\,260\,X_0 + 381\,050\,X_1 + 326\,850\,X_2 + 272\,640\,X_3 + 218\,430\,X_4 + 178\,040\,X_5 + 119\,890\,X_6 + 70\,775\,X_7 - 1\,944\,087 = 0.$$
$$306\,440\,X_0 + 268\,440\,X_1 + 230\,440\,X_2 + 192\,440\,X_3 + 154\,440\,X_4 + 119\,890\,X_5 + 92\,153\,X_6 + 50\,828\,X_7 - 1\,854\,752 = 0.$$
$$178\,480\,X_0 + 156\,550\,X_1 + 134\,610\,X_2 + 112\,660\,X_3 + 90\,720\,X_4 + 70\,750\,X_5 + 50\,828\,X_6 + 37\,683\,X_7 - 159\,775 = 0.$$

The values of the variables become:

$X_0 = +148.71$	$X_3 = -139.92$	$X_6 = +9.32$	$X_9 = +147.72$
$X_1 = +61.24$	$X_4 = -53.57$	$X_7 = -82.38$	
$X_2 = -36.48$	$X_5 = -110.26$	$X_8 = +55.72$	

(Case B).—Perfect Contact at Sill.

$$525\,440\ X_0 + 451\,010\ X_1 + 388\,880\ X_2 + 327\,170\ X_3 + 266\,080\ X_4 + 211\,180\ X_5 + 157\,050\ X_6 + 103\,790\ X_7 - 58\,716\,000 = 0.$$

$$451\,010\ X_0 + 408\,770\ X_1 + 342\,380\ X_2 + 288\,310\ X_3 + 234\,630\ X_4 + 186\,320\ X_5 + 138\,620\ X_6 + 91\,640\ X_7 - 49\,938\,000 = 0.$$

$$388\,880\ X_0 + 342\,380\ X_1 + 307\,990\ X_2 + 249\,430\ X_3 + 203\,190\ X_4 + 161\,460\ X_5 + 120\,180\ X_6 + 79\,476\ X_7 - 41\,019\,000 = 0.$$

$$327\,170\ X_0 + 288\,310\ X_1 + 249\,430\ X_2 + 222\,680\ X_3 + 171\,730\ X_4 + 136\,590\ X_5 + 101\,740\ X_6 + 67\,314\ X_7 - 32\,131\,000 = 0.$$

$$266\,080\ X_0 + 234\,630\ X_1 + 203\,190\ X_2 + 171\,730\ X_3 + 149\,180\ X_4 + 111\,720\ X_5 + 83\,305\ X_6 + 55\,154\ X_7 - 25\,820\,000 = 0.$$

$$211\,180\ X_0 + 186\,320\ X_1 + 161\,460\ X_2 + 136\,590\ X_3 + 111\,720\ X_4 + 98\,020\ X_5 + 66\,544\ X_6 + 44\,099\ X_7 - 18\,558\,000 = 0.$$

$$157\,050\ X_0 + 138\,620\ X_1 + 120\,180\ X_2 + 101\,740\ X_3 + 83\,305\ X_4 + 66\,544\ X_5 + 56\,585\ X_6 + 33\,044\ X_7 - 12\,931\,000 = 0.$$

$$103\,790\ X_0 + 91\,640\ X_1 + 79\,476\ X_2 + 67\,314\ X_3 + 55\,154\ X_4 + 44\,099\ X_5 + 33\,044\ X_6 + 28\,791\ X_7 - 5\,697\,700 = 0.$$

The values of the variables become:

$X_0 = +310.1$	$X_3 = -96.30$	$X_6 = -301.4$	$X_9 = +1\,615.6$
$X_1 = +184.4$	$X_4 = -74.07$	$X_7 = -593.4$	
$X_2 = +52.4$	$X_5 = -225.5$	$X_8 = -871.9$	

Discussion of Results.

The values of the variables, X_0, X_1, etc., previously obtained, give the reactions between the horizontal and vertical girders for the two gates and different conditions of sill contact, and $(P_0 + X_0)$, $(P_1 + X_1)$, etc., are the resultant girder loads per linear unit of leaf. Plate X gives these results in graphic form for the 77 ft. 6-in. and 49 ft. 6-in. gates, respectively.

On this plate, Figs. 1 and 6 show cross-sections of the leaf with its dimensions, moments of inertia, etc., also the total water pressure acting against the gate, and Figs. 3 and 8 give the reactions between the horizontals and the vertical girder, in pounds per linear foot of gate leaf, *B* being for no contact and *A* for perfect contact at sill.

Figs. 2 and 7 of Plate X give the resultant loads per linear foot on the different horizontals for the three different cases: *A*, vertical

stiffness and perfect sill contact; *B,* vertical stiffness but no sill contact; and *C,* no vertical stiffness at all.

Figs. 4 and 9 of Plate X show the deflections of the miter posts, that is, the distance they move down stream parallel with the axis of the lock for the three cases just mentioned. These last curves are in general agreement with those showing the resultant girder loads.

Loads on Horizontals.—The curves show that, with no contact at the sill, the values of X (reactions of vertical against horizontal girders) are quite small, so that, except at the very top, the deviations from a purely hydrostatic loading are inconsiderable and due to accidental causes. With "perfect" contact, the girders in the lower portion of the leaf (from one-third to one-half the height) were relieved of a large part of their hydrostatic load, the horizontals higher up receiving a proportionately greater loading. The girders closest to the top showed, in all cases, the largest proportional increase. In the 77 ft. 6-in. gate, there was also an increase for the horizontals in the middle third of the height. As a whole, however, the effect of the vertical stiffness was decidedly greater for the 49 ft. 6-in. gate.

In proportioning the Panama gates, it was decided to use a load corresponding to a head of 20 ft. for all girders within 20 ft. of the top. For those lower down, the hydrostatic head was taken, but, in gates more than 75 ft. high, the girders in the middle third of the height had this load increased by from 5 to 10 per cent.

For smaller gates, the effect of vertical stiffness would probably be greater than for the very high and rather thin Panama gates.

There is no reason, however, to doubt that the common assumption of hydrostatic loading, except for a few girders near the top, will give safe results. In the lower part of the leaf, the stresses in actual service will generally be quite small in a gate designed for the hydrostatic head, as it is very probable that there will be some contact at the sill. However, the increase in cost involved, is not great, and in any event the miter gate will weigh much less than any form of caisson or single-leaf gate.

Reaction of Gate Against Sill.—For the case of no contact, the reaction, of course, is equal to zero. For "perfect contact", the sill pressure is equal to the end reaction, X, at the bottom of the vertical girder plus the direct water load, P, on the lowest arch.

These values for the two gates are as follows: (See Figs. 2 and 7 of Plate X).

For the 77 ft. 6-in. gate,

$$12\,(P_{15} + X_{15}) = 9\,560 + 27\,322 = 36\,882 \text{ lb. per lin. ft.}$$

or, $$\frac{36\,882}{184\,875} = 19.4\% \text{ of the total load of the gate.}$$

For the 49 ft. 6-in. gate,

$$12\,(P_9 + X_9) = 7\,474 + 19\,387 = 26\,857 \text{ lb. per lin. ft.}$$

or, $$\frac{26\,857}{76\,565} = 35.1\% \text{ of the total load.}$$

It will be noted that the total sill reaction is proportionately greater for the lower gate.

In case there is some elastic movement of the sill under pressure, the sill reaction and also the loads on horizontals and the deflections will be values intermediate between Cases *A* and *B*.

As stated previously, it is believed that Case *A* is a sufficient maximum.

For small locks it is not unusual to make the masonry sill strong enough to withstand the theoretical maximum of $66\frac{2}{3}\%$ of the total water pressure acting against the gate. For the large proportions at Panama, it would have been difficult to make the sill walls strong enough to carry this maximum, and it seemed entirely unnecessary. It was deemed quite safe to assume a pressure of 50 000 lb. per lin. ft. of sill or about one-quarter of the whole load on the gate.

Stresses in the Vertical Bracing.—This bracing corresponds to the vertical girder in the computations, the loads acting on it being the forces, X_0, X_1, etc., applied transversely at distances corresponding to the spacing of the horizontal girders.

The chord stresses are readily obtained from the values shown on Plate X. The unit stresses were found to be not more than 4 500 lb. per sq. in. in any part of the bracing.

The shears for proportioning the web thickness and the rivet connections were obtained in a similar manner from the transverse forces, X_0, X_1, etc.

APPENDIX

Method for the Solution of Simultaneous Equations.

For the solution of simultaneous equations of the first degree, the method originated by the celebrated astronomer, Gauss, is probably the best for practical use in the engineer's office. The solution by determinants is theoretically attractive, but its application is not satisfactory. Graphical methods are the best in some cases, but can hardly be used where the coefficients of the different variables differ widely in magnitude, and where a high degree of accuracy is required.

Gauss' method in a simple form may be stated as follows:

Given, say, three simultaneous equations of the form:

$$a\,x + b\,y + c\,z + m = 0 \quad\ldots\ldots\ldots\ldots (1)$$
$$b\,x + d\,y + e\,z + n = 0 \quad\ldots\ldots\ldots\ldots (2)$$
$$c\,x + e\,y + f\,z + p = 0 \quad\ldots\ldots\ldots\ldots (3)$$

to find the values of x, y, and z.

From Equation (1), we obtain:

$$x = -\frac{b\,y + c\,z + m}{a}$$

and substituting in Equations (2) and (3), we have:

$$\left(d - \frac{b}{a}\,b\right) y + \left(e - \frac{b}{a}\,c\right) z + \left(n - \frac{b}{a}\,m\right) = 0 \ldots (2^1)$$
$$\left(e - \frac{c}{a}\,b\right) y + \left(f - \frac{c}{a}\,c\right) z + \left(p - \frac{c}{a}\,m\right) = 0 \ldots (3^1)$$

which may be written,

$$d_1\,y + e_1\,z + n_1 = 0 \ (2^1)$$
$$e_1\,y + f_1\,z + p_1 = 0 \ (3^1)$$

In these two equations the coefficients are the same as in Equations (2) and (3), with a subscript added.

From Equations (2^1) and (3^1), we have, in the same way,

$$\left(f_1 - \frac{e_1}{d_1}\,e_1\right) z + \left(p_1 - \frac{e_1}{d_1}\,n_1\right) = 0 \ldots\ldots\ldots (3^2)$$

which may be written,

$$f_2\,z + p_2 = 0 \ldots\ldots\ldots\ldots (3^2)$$

From Equation (3^2), we obtain the value of $z = -\dfrac{p_2}{f_2}$, and the value of x and y by substituting in the first and second sets of equations.

By writing the equation in tabular form, the relation of the successive coefficients is made clearer, and a valuable check on the arithmetical work is obtained at each step.

TABLE 1.

— —	x	y	z	—	—
Equation (1)............	a	b	c	m	$q = a + b + c + m$
Equation (2)............	b	d	e	n	$r = b + d + e + n$
Equation (3)............	c	e	f	p	$s = c + e + f + p$

TABLE 2.

— —	y	z	—	—
Equation (2^1)................	d_1	e_1	n_1	r_1
Equation (3^1)................	e_1	f_1	p_1	s_1

TABLE 3.

—	z	—	—
Equation (3^2)................	f_2	p_2	s_2

On examining Equations (2^1) and (3^1), it will be seen that the coefficients, d_1, e_1, etc., are always of the form,

$$T - \frac{U\,V}{H},$$

in which,

T = the corresponding coefficient in Equations (2) and (3), that is, the coefficient of the same variable, omitting the subscript;

U = the coefficient in the top horizontal line, that is, in Equation (1) vertically above T;

V = the last coefficient to the left in the same horizontal line with T; and

H = the coefficient of x in the top row.

The values of q, r, and s, in the last column of Table 1 are written down by simply adding the preceding coefficients in each horizontal row.

In Tables 2 and 3, r_1, s_1, s, like the coefficients, d_1, e_1, n_1, etc., are obtained by the formula, $T - \frac{U\,V}{H}$, previously given.

The check consists in adding up each horizontal line; the last term should be equal to the sum of the preceding ones, that is, for instance, s should equal $e_1 + f_1 + p_1$.

Numerical Example.

$$482.5\,x + 348.4\,y + 238.6\,z + 140.7\,u - 48\,915 = 0$$
$$348.4\,x + 280.5\,y + 186.0\,z + 111.4\,u - 37\,167 = 0$$
$$238.6\,x + 186.0\,y + 146.9\,z + 82.0\,u - 24\,484 = 0$$
$$140.7\,x + 111.4\,y + 82.0\,z + 66.0\,u - 11\,760 = 0$$

TABLE 4.

x	y	z	u	——	——
482.5	348.4	238.6	140.7	— 48 915	— 47 704.8
348.4	280.5	186.0	111.4	— 37 167	— 36 240.1
238.6	186.0	146.9	82.0	— 24 484	— 23 330.5
140.7	111.4	82.0	66.0	— 11 760	— 11 359.9

TABLE 5.

y	z	u	——	——
28.92	13.70	9.80	— 1 847	— 1 794.5
13.70	28.90	12.42	— 296	— 2 410.0
9.80	12.42	24.97	+ 2 503	+ 2 550.1

TABLE 6.

z	u	——	——
22.41	7.78	+ 579	+ 609.2
7.78	21.65	+ 3 128.9	+ 3 158.3

TABLE 7.

	u	——	——
	18.95	+ 2 927.9	+ 2 946.84

The coefficients for the first equation in Table 5, will be,

$$280.5 - \frac{348.4 \times 348.4}{482.5} = 28.92$$

$$186.0 - \frac{238.6 \times 348.4}{482.5} = 13.70$$

$$111.4 - \frac{140.7 \times 348.4}{482.5} = 9.80$$

$$-37\,167 - \frac{48\,915 \times 348.4}{482.5} = -1\,847$$

and, as a check, the last term

$$\left(-36\,240.1 - \frac{47\,704.8 \times 348.4}{482.5} \right) = -1\,794.5$$

which should and does equal the sum of the preceding terms.

The roots are $x = +58$, $y = +103$, $z = +27$, and $u = -154$.

The computations were carried out mainly with the use of the well-known cylindrical slide-rule of Edwin Thacher, M. Am. Soc. C. E., and the "Millionaire" multiplying machine.

Although such operations are necessarily somewhat tedious, the set of equations containing fourteen variables was solved by two computers in less than 15 hours of actual work.

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PAPERS AND DISCUSSIONS

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AIR TANKS ON PIPE LINES*

By Minton M. Warren, Assoc. M. Am. Soc. C. E.

Synopsis.

The object of this paper is to call attention to the utility of air tanks on long pipe lines and to some of their practical disadvantages, to formulate the theoretical principles involved, and to derive simple formulas which may be used as a guide to determine their safe dimensions.

The contents of this paper may be briefly classified as follows:

Formulas for air tank design;

Derivation of formulas;

Practical questions of design and operation; and

Numerical examples.

Conclusions.—Although there is a general idea that air tanks are not successful for regulating purposes on pipe lines, and that their design is a matter of great uncertainty, it can be shown that, if properly built, they are of great practical value in improving regulation and preventing water-hammer, and their design is simple and based on fundamental laws.

The literature on the design of air tanks on pipe lines for water-wheel regulation is very limited. This is, perhaps, due to two causes: first, because of a general belief that the problem is so complex as to need a difficult mathematical solution by calculus; second, because

* This paper will not be presented for discussion at any meeting of the Society, but written communications on the subject are invited for subsequent publication in *Proceedings*, and with the paper in *Transactions*.

of a popular idea that, in order to be effective, the air tank must be so large as to be prohibitive in cost. Neither of these, however, is true. The design of such a tank is simple, and comparatively small tanks are in commercial operation on pipe lines, and have greatly improved the regulation and reduced the trouble from water-hammer.

For the sake of convenience the formulas are given first, the assumptions used in their derivation are then listed, and finally they are derived from Newton's second law of motion and the physical laws governing the expansion and compression of air.

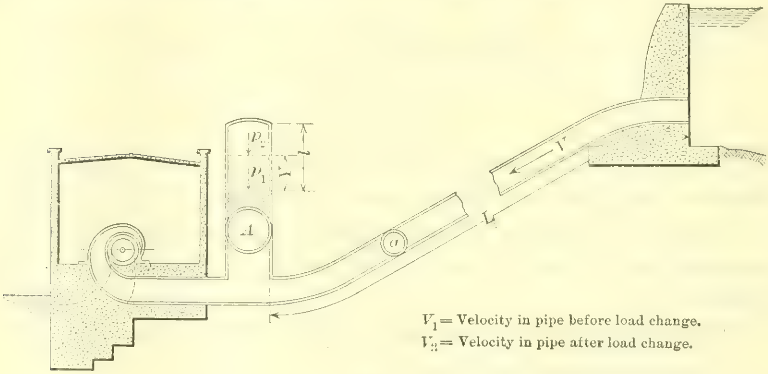

Fig. 1.

More complicated formulas are appended which are derived by calculus with fewer assumptions.

Nomenclature.—The following gives the meaning of every term used in deriving the formulas. All units are in feet, pounds, or seconds.

A = cross-sectional area of air tank, in square feet;

g = acceleration of gravity = 32.2 ft. per sec. per sec.;

$K = \frac{L\,a}{g\,A\,p_1}(V_1 - V_2)^2$;

K_1 = Head lost between air tank and pipe line, in feet, when $a\,(V_1 - V_2)$ cu. ft. per sec. are flowing into tank;

L = length of pipe, in feet, between open reservoir or fore-bay and air tank;

l = length of air column in tank before load change, in feet;

a = cross-sectional area of pipe, in square feet;

p_1 = air pressure in air tank before load change, including atmospheric pressure, measured in feet of water;

p_2 = maximum or minimum air pressure in air tank due to load change, including atmospheric pressure, measured in feet of water;

t = time for pressure in air tank to change from p_1 to p_2, in seconds;

V_1 = velocity of water in pipe before load change, in feet per second;

V_2 = velocity of water in pipe after load change, in feet per second;

v = velocity of water in pipe at any time, in feet per second;

W = weight of 1 cu. ft. of water = 62.4 lb.;

Y = maximum rise or fall of water in air tank, in feet, measured from water level before load was changed;

y = rise or fall of water in air tank at any time.

Formulas.—The following formulas are sufficiently accurate for most practical cases. The true values of Y and p_2 will lie between the values obtained for isothermal* and for adiabatic* compression or expansion, but will be nearer the latter values.

Assuming isothermal compression or expansion:

$$Y = \sqrt{K\,l + \frac{K^2}{4}} \mp \frac{K}{2} \qquad (A)$$

$$\text{where } K = \frac{L\,a}{g\,A\,p_1}\,(V_1 - V_2)^2$$

$$p_2 = \frac{p_1\,l}{l \mp Y} \qquad (B)$$

Assuming adiabatic compression or expansion:

$$(l - Y)^{1.41} = \frac{Y\,l^{1.41}}{K + Y} \qquad (C)$$

for loads thrown off, and

$$(l + Y)^{1.41} = -\frac{Y\,l^{1.41}}{K - Y} \qquad (C_1)$$

for loads thrown on.

$$p_2 = p_1\left(\frac{l}{l \mp Y}\right)^{1.41} \qquad (D)$$

---

* Isothermal compression or expansion assumes that the temperature of the air in the tank remains constant. All the heat generated in compressing the air, therefore, must escape instantly through the walls of the tank and into the water.

Adiabatic compression or expansion assumes that no heat passes through the tank walls or between the air and water.

Where plus and minus signs appear, the minus sign applies to loads thrown off, the plus sign to loads thrown on.

Assumptions.—These formulas are derived by using the following assumptions, and, before using them on any specific case, these assumptions should be examined in order to make sure that they apply to the case in question.

1.—Pressure in tank rises or falls at a constant rate;*

2.—Water level in tank rises or falls at a constant rate;*

3.—Water pressure due to change of water level in tank is neglected.*

4.—No loss of head between tank and pipe line;

5.—Time necessary to open or close wheel gates is neglected;

6.—Friction in pipe line is neglected.

7.—Time necessary for a pressure wave to travel the length of the pipe is neglected.

8.—Governor action is neglected.

The errors introduced by the use of these assumptions offset each other to a large extent, as some of them will lead to values of p_2 which are too large, and others to values which are too small. It should be noted, however, that if the loss of head between the tank and pipe line is comparatively large, the pressure may rise higher in the pipe line and wheel than in the tank.

In general, the error caused by Assumption 6 is negligible, as the rise in air pressure is large in comparison with the difference in friction head at any two loads. In order to be absolutely safe, this friction head may be added to the value of p_2 obtained from the formulas for loads thrown off, but this will give results which are too large. The reverse is true for loads thrown on.

Assumption 7 is reasonable, as the time for a rise of pressure to be transmitted along the pipe line is usually small in comparison with the time of oscillation in the air tank.

The effect of governor action (Assumption 8) is discussed later.

From numerical computations it appears that Assumptions 1, 2, and 3, taken together, lead to values of p_2 which are too small.

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* Formulas (P) and (Q), derived by calculus, are appended; they are deduced without the use of Assumptions 1, 2, and 3. These may be used for the final design of a tank.

Derivation of Formulas.—To avoid confusion, the formulas for load thrown off (air compressed) are first derived.

Using the fundamental law,

$$\text{Mass} \times \text{Acceleration} = \text{Force},$$

and multiplying through by time, we get

$$\text{Mass} \times \text{Change in Velocity} = \text{Force} \times \text{Time} \ldots\ldots\ldots (E)$$

The mass of the water in the pipe is $\frac{W\,a\,L}{g}$. When a load is thrown off, the velocity is cut down from V_1 to V_2, and the change of velocity, therefore, is $(V_1 - V_2)$.

The force tending to slow down the water in the pipe is due to the difference in head at the two ends, or rise of pressure at the lower end. Assuming no head lost between the air tank and pipe line (Assumption 4), and neglecting the pressure due to the rise of water in the tank (Assumption 3), this difference in head is equal to the rise of air pressure in the tank, which varies from zero at the beginning to $(p_2 - p_1)$ at the end. Assuming the rise to be at a constant rate (Assumption 1), the average force tending (during the time, t) to slow down the water will be $W\,a\,\frac{p_2 - p_1}{2}$, in pounds.

As a matter of fact, the average force is less than this, as it rises faster at the end than at the beginning.

Using Formula (E), and inserting the foregoing values, we get

$$\frac{W\,a\,L}{g} \times (V_1 - V_2) = W\,a\,\frac{p_2 - p_1}{2} \times t \ldots\ldots\ldots\ldots (G)$$

The time, t, is unknown. Neglecting the short time necessary for the governor to move the gates and decrease the quantity of water supplied to the wheel (Assumption 5), the net quantity of water coming into the tank when the load is first thrown off will be the volume flowing in from the pipe line ($a\,V_1$) less the volume flowing out to the wheel ($a\,V_2$). Therefore the water will start to rise in the tank at a velocity of $\frac{a}{A}(V_1 - V_2)$, in feet per second, and this will decrease to zero in the time, t. Assuming this decrease to be at a constant rate (Assumption 2), the average velocity will be half of this, or $\frac{a}{2\,A}(V_1 - V_2)$. As a matter of fact, the average velocity is

greater than this, for the velocity decreases at a slower rate at the end of the time than at the beginning.

If the water rises in the tank at an average velocity of $\frac{a}{2\,A}(V_1 - V_2)$, in feet per second, it will take $\frac{2\,A\,Y}{a\,(V_1 - V_2)}$ seconds to rise a distance, Y. Therefore,

$$t = \frac{2\,A\,Y}{a\,(V_1 - V_2)} \qquad (H^*)$$

Inserting this value of t in Formula (G), we get

$$\frac{W\,a\,L}{g}(V_1 - V_2) = W\,a\,\frac{(p_2 - p_1)}{2}\,\frac{2\,A\,Y}{a\,(V_1 - V_2)}.$$

Simplifying,

$$(p_2 - p_1)\,Y = \frac{L\,a}{g\,A}(V_1 - V_2)^2 \qquad (I)$$

From this, Formulas (A) or (C) can be deduced by using the laws of thermodynamics which apply to the compression or expansion of gases. In order to get the relation between the two unknown quantities, p_2 and Y, it is necessary to know the variation in temperature in the air tank.

If the compression took place very slowly, and all the heat generated had time to escape into the water and through the walls of the tank, the isothermal relation would hold, and

$$\frac{p_2}{p_1} = \frac{\text{original volume of air in tank}}{\text{volume of air in tank after compression}} = \frac{A\,l}{(l - Y)\,A}$$

or

$$p_2 = \frac{p_1\,l}{l - Y} \qquad (B)$$

If the compression took place very quickly, and all the heat of compression were retained, the adiabatic relation would hold, and

$$\frac{p_2}{p_1} = \left(\frac{l}{l - Y}\right)^{1.41} \qquad (D)$$

As a matter of fact, the true condition will be somewhere between the two. With full load thrown off, the isothermal assumption will give results which are much too small, and in general the adiabatic assumption is much nearer the truth.

---

* This formula, by itself, will give a value of t which is too large. See Formula (R).

For instance, in one actual case, for full load thrown off, the compression would take less than 5 sec., and, assuming adiabatic compression, the temperature would rise more than 100° Fahr. It is apparent that in this short time not much of the heat would escape, and the isothermal assumption, therefore, gives results about 14% too small.

Although formulas for isothermal compression give values of p_2 (maximum pressure) which are too small, they give larger values of Y (rise of water in the tank) than formulas for adiabatic compression.

Isothermal Compression.—Assuming, first, isothermal compression: Combining Formulas (I) and (B),

$$\frac{L\,a}{g\,A}(V_1 - V_2)^2 = p_1\left(\frac{l}{l - Y} - 1\right)\,Y = \frac{p_1\,Y^2}{l - Y} \quad \ldots\ldots\ldots (J)$$

let

$$K = \frac{L\,a}{g\,A\,p_1}(V_1 - V_2)^2 \quad \ldots\ldots\ldots\ldots\ldots\ldots (M)$$

and insert in Formula (J).

$$K = \frac{Y^2}{l - Y}$$

$$K\,l - K\,Y = Y^2$$

solving

$$Y = \sqrt{K\,l + \frac{K^2}{4}} - \frac{K}{2} \quad \ldots\ldots\ldots\ldots\ldots\ldots (A)$$

Adiabatic Compression.—Assuming, second, adiabatic compression: Combining Formulas (I) and (D),

$$\frac{L\,a}{g\,A}(V_1 - V_2)^2 = p_1\left[\left(\frac{l}{l - Y}\right)^{1.41} - 1\right]Y \quad \ldots\ldots\ldots (N)$$

Inserting K, as before,

$$K = \left(\left(\frac{l}{l - Y}\right)^{1.41} - 1\right)Y$$

or

$$(l - Y)^{1.41} = \frac{Y\,l^{1.41}}{K + Y} \quad \ldots\ldots\ldots\ldots\ldots\ldots (C)$$

This must be solved by trial and error. As a first trial, a value of Y slightly smaller than that obtained from Formula (A) may be used.

Load Thrown On.—In the case of a load thrown on, exactly the same reasoning applies, and Y, being measured down instead of up, will be a negative quantity. When the numerical value is inserted, the signs, therefore, must be changed as indicated.

Full Load Changes.—When full load is thrown off, the same formulas apply, and, as V_2 equals zero, Formula (I) becomes

$$(p_2 - p_1)\ Y = \frac{L\ a\ V_1^2}{g\ A}.$$

This can be checked easily by applying the theorem of work and energy, using the assumptions previously listed. All the water is stopped, and all the energy of the moving water is expended in compressing the air in the air tank.

Work of compressing air in tank = Energy of water in pipe line.

$$Y \times \frac{W A\ (p_2 - p_1)}{2} = W\ a\ L \times \frac{V_1^2}{2\ g}.$$

Simplifying,

$$(p_2 - p_1)\ Y = \frac{L\ a\ V_1^2}{g\ A}.$$

Similarly, for full load thrown on, V_1 becomes zero.

Practical Questions of Design, Surges, Etc.—In actual practice it has sometimes happened that an air tank made the regulation and surges in the pipe line worse instead of better. This has been due to surges which were set up and either aggravated by governor action or continued naturally for a considerable time before being damped out by friction.

These surges can be broken up by using some sort of a differential device which restricts the entrance to the tank, or a valve which causes the water to flow into the tank at a different rate from that at which it flows out.

Mr. R. D. Johnson has invented and patented a device which not only eliminates surges, but cuts down the size of the tank required.

In one hydro-electric development where trouble was caused by surges, a flap-valve surrounded by orifices was placed in the entrance to the air tank. When load was thrown on, the flap opened and allowed the water to flow out freely. When the water started to flow back into the tank, the flap closed and the water was forced through the orifices, with a consequent loss of head. This broke up the surges, and there was no more trouble.

In one case, under normal operating conditions, an air tank of moderate dimensions placed near the power-house decreased the sudden variations of pressure due to changing load by about two-thirds.

In order to keep the tank full of air, it is necessary to replenish the supply from time to time from a compressor or other source, as a certain quantity is apt to be absorbed by the water or leak through the tank. If the air should entirely leak out, the tank would not only become useless for regulating purposes, but might fail on account of dangerous pressures due to water-hammer.

The larger the volume of air in the tank, the more efficient will it be for regulating purposes. Care must be taken, however, not to fill the tank so full of air that when a sudden load is thrown on, all the water in the tank will be exhausted and the wheel will suck air. Automatic safety devices may be put on to prevent this, and gauges on the side of the tank will indicate the water level. It is conceivable that in some cases air might be given out by the water and fill the tank too full. The writer knows of no such case, however, although the reverse is common.

In most air tanks the compression due to load changes will take place in so short a time that little of the heat will have time to flow through the walls of the tank or into the water, and for this reason the formulas for adiabatic compression will give results nearer the truth. In any case, they are more conservative to use.

Other Formulas.—In order to get formulas without the use of Assumptions 1, 2, and 3, it is necessary to use the calculus. The other assumptions (4, 5, 6, 7, and 8) are used.

The net quantity of water flowing into the tank is equal to that flowing in minus that flowing out; therefore,

$$A \frac{d\,y}{d\,t} = a\,(v - V_2)$$

Mass $\times$ Acceleration $=$ Force

therefore (for load thrown off),

$$\frac{W\,a\,L}{g} \times \frac{d\,v}{d\,t} = -\,(p - p_1 + y)\,W\,a.$$

Using these two formulas in conjunction with Formulas (B) and (D) (the isothermal and adiabatic relation), we can get a differential equation for $\frac{d\,y}{d\,t}$ in terms of y and $d\,y$ which can be integrated by using

the fact that when $y = 0, \frac{d\,y}{d\,t} = \frac{a}{A}(V_1 - V_2)$; and when $y = Y$, $\frac{d\,y}{d\,t} = 0$.

By this integration we get the following:

Isothermal.

For load off:

$$Y^2 - 2\,p_1\left(l \log._e \left(1 - \frac{Y}{l}\right) + Y\right) = p_1\,K \text{ . . . . } (P)$$

For load on:

$$Y^2 - 2\,p_1\left(l \log._e \left(1 + \frac{Y}{l}\right) - Y\right) = p_1\,K \text{ . . . . } (P_1)$$

Adiabatic.

For load off:

$$Y^2 - 2\,p_1\left[\frac{l}{0.41} - \frac{l^{1.41}}{0.41\,(l - Y)^{0.41}} + Y\right] = p_1\,K \text{ . . . . } (Q)$$

For load on:

$$Y^2 - 2\,p_1\left[\frac{l}{0.41} - \frac{l^{1.41}}{0.41\,(l + Y)^{0.41}} - Y\right] = p_1\,K \text{ . . . . } (Q_1)$$

A formula for t, derived by calculus, using all the assumptions except 2, is:

$$t = \frac{\pi}{2}\sqrt{\frac{L\,A\,Y}{a\,g\,(p_2 - p_1)}} \text{ . . . . } (R)$$

This t is the time required for Y to reach a maximum. It is the time for a quarter of the cycle, and, after the water has risen in the tank (with load thrown off), it will, like a pendulum, fall to the starting point again, and on below it to almost the same extent; then rise again, and so on until damped out by friction. It is to prevent this surge that valves or differential devices must be used. Formula (R) will give a value of t which is too small. The true value will be between this and the value obtained from Formula (H).

Where it is desired to compute the heat generated or lost by adiabatic compression or expansion, the following formula may be used.

$$\frac{T_2}{T_1} = \left(\frac{l}{l \mp Y}\right)^{0.41}.$$

The minus sign is for load off, the plus sign for load on.

T_1 is the absolute temperature before start of rise or fall;

T_2 is the absolute temperature at maximum point of rise or fall.

If the Fahrenheit scale is used, the absolute temperature may be obtained by adding 459° to the temperature registered on the ordinary thermometer.

Head Lost in Entering Air Tank.—If the pipe line runs into the air tank on one side and out on the other, the pressure in the pipe line will at all times be equal to the pressure in the air tank (Assumption 4).

When a load is suddenly thrown off, the quantity of water flowing out of the tank will be suddenly decreased, but the quantity flowing in will not be changed until the water has risen and the air pressure consequently increased. This increase of pressure causes a difference of head, between the two ends of the pipe line, which slows down the water. It increases from 0 to $(p_2 - p_1)$ and is greatest at the time when the water has been slowed down to its new velocity.

It is obvious that the air tank would be more efficient if the pressure could be made to rise instantly and act on the water at the time when it is most needed, or before the velocity has been checked.

By causing a loss of head between the air tank and the pipe line, this condition can be approached. If the loss of head is made too great, however, the pressure may rise instantly to a greater value than it would have reached with the simple tank, and this is all the more undesirable as the sudden rise causes water-hammer and is more difficult for the governors to handle than the gradual rise.

A well-designed orifice, therefore, would cause about the same loss of head at the beginning as the compressed air would cause at the end.

An approximate formula can be worked out by computing the maximum loss of head between the pipe line and the air tank at the start of the rise. This will be

$$C \frac{(V_1 - V_2)^2}{2\,g},$$

where C is a constant depending on the shape and size of the entrance to the tank.

Calling this loss of head K_1, and assuming that the difference in head varies at a constant rate from this value at the beginning to $(p_2 - p_1)$ at the end, we get an average force of $\frac{K_1 + p_2 - p_1}{2}$, in pounds, tending to slow down the water.

Inserting this value in Formula (G) and proceeding as before, we get for isothermal compression:

$$(K_1 - p_1)\,Y^2 - (Kp_1 + K_1\,l)\,Y = - Kp_1\,l \ldots\ldots (S)$$

For load thrown on, this becomes

$$(K_1 + p_1)\,Y^2 + (K_1\,l - Kp_1)\,Y = Kp_1\,l \ldots\ldots\ldots (S_1)$$

Where $K_1 = 0$, these become Formula (A).

A similar formula can be worked out for adiabatic compression. When the calculus is used and Assumptions 1, 2 and 3 are omitted, a differential equation is derived which cannot be integrated.

Considering the many uncertainties of the problem, the foregoing formula is probably sufficiently accurate, but it should be used with the full realization of the assumptions made in its derivation.

Numerical Example.—In order to show the comparative variation of the different formulas, the following numerical case has been worked out for full load thrown off and on. The dimensions are taken from an actual plant, and an extreme velocity of 12.5 ft. per sec. is taken to illustrate more clearly the differences in the various formulas.

$l = 52$ ft.;
$L = 2\,100$ ft.;
$a = 44.2$ sq. ft. (pipe 7.5 ft. in diameter);
$A = 77$ sq. ft. (two tanks 7 ft. in diameter);
$p_1 = 470$ ft.
$V_1 = 12.5$ ft. per sec.;
$V_2 = 0$.

Table 1 shows the values obtained from the various formulas. In studying these results, it should be noted that the difference in the results of the formulas for adiabatic and isothermal compression is 14 per cent. The difference between the results obtained by the more approximate Formulas (A) and (C) and the results obtained from Formulas (P) and (Q) is from 3 to 6% for this case.

TABLE 1.

	Formula.	Y, in feet.	p_2, in feet.	Percentage of normal pressure (p_1).
Full load off:				
Isothermal............	(A) and (B)	20.0	764	163
	(P)	20.8	783	166
Adiabatic..................	(C) and (D)	16.9	818	174
	(Q)	17.7	845	180
Full load on:				
Isothermal..................	(A) and (B)	32.5	290	62
	(P_1)	27.5	307	65
Adiabatic..................	(C_1) and (D)	27.6	258	55
	(Q_1)	23.8	276	59

For load thrown on, Formulas (P_1) and (Q_1) give results 6% apart for the two relations; and the more approximate Formulas (A) and (C_1) give results differing from Formulas (P_1) and (Q_1) by from 3 to 4 per cent.

The error caused by assuming either the adiabatic or isothermal relation, therefore, may be greater than the error caused by using the simpler Formulas (A) and (C) and so, taking into account the other assumptions, it would seem that Formulas (A) and (C) are sufficiently accurate for most practical cases, although a final check may be made by Formulas (P) and (Q).

If a restricted orifice were inserted between the pipe line and air tank which caused a loss of head of 190 ft. with the full load flow ($a\ V_1$) entering the tank the results would be as follows:

	Formula	Y	p_2	Percentage of p_1
Full load off: ($K_1 = 190$)	(S)	15.3	667	142
Full load on: ($K_1 = 190$)	(S_1)	18.7	346	74

A comparison of these figures with those given for the simple tank shows that the use of an orifice makes the air chamber a great deal more effective. It also reduces the possibility of trouble from surges.

The foregoing formulas have not been published before, as far as the writer knows, with the exception of Formula (P) which was deduced by Mr. Johnson in the article mentioned below.

In studying the effect of differential devices and friction in air tanks and surge tanks, the following references will be found useful:

"The Surge Tank in Power Plants." By R. D. Johnson. *Transactions,* Am. Soc. Mech. Engrs., Vol. 30 (1908), page 833.

"The Differential Surge Tank." By R. D. Johnson. *Transactions,* Am. Soc. C. E., Vol. LXXVIII (1915), page 760.

"Penstock and Surge Tank Problems." By Minton M. Warren. *Transactions,* Am. Soc. C. E., Vol. LXXIX (1915), page 238.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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PAPERS AND DISCUSSIONS

THE CAPE COD CANAL

By William Barclay Parsons, M. Am. Soc. C. E.

To be Presented October 3d, 1917.

Synopsis.

The Cape Cod Canal joins Cape Cod Bay with the waters adjacent to Long Island Sound, traversing the narrow isthmus of Cape Cod. It has a length of 8 miles, with dredged approach channels 5 miles long. The minimum width of the canal is 100 ft., the maximum, 300 ft., and the depth at low water, 25 ft.

This canal was suggested for commercial and naval use 300 years ago. The project, originally a colonial one, subsequently became a national one, and finally was carried out by private capital. It has been in successful operation since the summer of 1914, and is now used by commercial and naval vessels.

Part I of this paper is devoted to history and location, and contains considerable data on construction. Part II is devoted entirely to hydraulics. The canal is the largest open artificial waterway connecting two seas having non-synchronous tides.

Note.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and, when finally closed, the papers, with discussion in full, will be published in *Transactions*.

PART I.

The southeast portion of the State of Massachusetts is a curiously shaped, narrow, hooked point enclosing nearly three-quarters of a circle having a radius of about 12 miles. The south side of the enclosing land is extended southwestward to another point at Woods Hole, which, with a succession of islands, makes the barrier separating Buzzards Bay from Vineyard Sound. To this whole area of land the name of Cape Cod is applied, with Provincetown at the north, where the Pilgrims in the *Mayflower* made their first landing in 1621. The material composing the peninsula is sand, gravel, and granite boulders—an old glacial terminal moraine. The sand has been acted on by the currents of the Gulf Stream flowing east and the eddy of the Arctic Current sweeping down the New England coast, and this accounts for the peculiar contour of the shore. To the south and southeast of Cape Cod are various irregular shoals, known as Nantucket Shoals, the outer limit of which, in 30 fathoms of water, is marked by the Nantucket Light Vessel, distant from Monomoy Point, the extreme southernmost part of Cape Cod, 59 geographical or 68 statute miles in an almost due south direction.

These shoals of shifting sand, over which the depth varies from less than 1 fathom to 25 fathoms, discovered first by Verrazzano in 1524, and named by De Mont in 1605 "Mallebarre", have always been a terror to navigators. The extent, depth of water, and character of the Nantucket Shoals have been described in a report by Lt.-Col. (now Col.) J. C. Sanford, Corps of Engineers, U. S. A., to the Chief of Engineers, under date of November 16th, 1909, from which the following extracts have been taken:

"The numerous and extensive shoals lying eastward and southeastward of the eastern entrance to Nantucket Sound and southward and eastward of the southeasterly elbow of Cape Cod, constitute probably the greatest danger to navigation to be found on any of the coastwise routes of the Atlantic coast of the United States north of Hatteras. In view of the numerous vessels passing around these shoals they are probably a greater menace to navigation than Hatteras. Their dangerous character is shown both by the large number of wrecks annually occurring there and by the large number of light vessels and other aids to navigators traversing the shoals.

"From Nantucket Sound to the ocean two channels lead through the shoals. The north or Pollock Rip Channel is the most used, as it is shorter, is somewhat protected from easterly storms by the

shoals outside it, and is closer to the shore; but it is quite circuitous and narrow in places and the tidal currents are strong and varying in direction. The second or south channel leads through the shoals in a nearly due east direction from Nantucket (Great Point) Lighthouse. It is somewhat deeper than the Pollock Rip Channel and much wider, but it is not so direct for coastwise vessels and carries a vessel much farther from the shore. This channel is considered in the United States Coast Pilot for the Atlantic coast as the dividing line between Nantucket and Monomoy Shoals, the shoals lying to the northward of the channel being called the Monomoy Shoals, while those to the southward are called the Nantucket Shoals. The following description of the Monomoy Shoals in general and of the shoals particularly named in the river and harbor act, with others lying along the course of the proposed improvement, is taken from the above publication:

"'Monomoy Shoals consist of numerous detached shoals of a shifting character with 3 to 18 feet over them, extending about 5½ miles in an easterly and 9½ miles in a southerly and south-southeasterly direction from Monomoy Point. Many parts of these shoals separated from others by narrow slues have special names and are briefly described below:

"'Bearse Shoal is the western and Pollock Rip the eastern part of the shoal extending from ⅜ mile to 3¾ miles eastward of Monomoy Lighthouse. These shoals consist of a series of sand shoals and sand ridges, with 4 to 18 feet over them and deep water between them.

"'Broken Part of Pollock Rip, with depths of 15 to 18 feet over it, lies eastward of Pollock Rip, and is separated from it by Pollock Rip Slue, which has a width of about ½ mile and depth of 3½ to 6 fathoms.

"'Twelve Foot Shoal, southward of the broken part of Pollock Rip, has 14 to 18 feet over it and lies 5½ miles SE ½ E from Monomoy Lighthouse.

"'Stone Horse Shoal, Little Round Shoal, and Great Round Shoal are portions of a continuous series of sand shoals and sand ridges with depths of 5 to 18 feet over them, lying directly eastward of the entrance of Nantucket Sound and between the two main channels. Stone Horse Shoal and Little Round Shoal lie on the south side of the deepwater channel between them and Pollock Rip. Great Round Shoal lies from 6 to 9½ miles in SSE direction from Monomoy Point Lighthouse; southward and eastward of this shoal for a distance of about 2½ miles there are numerous shoal spots with depths varying from 17 to 18 feet over them.

"'Shovelful Shoal, extending ¾ mile southward from Monomoy Point, is bare in places and rises abruptly from the deep waters of Butlers Hole.

"'Handkerchief Shoal is the extensive shoal, with from 3 to 18 feet over it, lying southwestward of Monomoy Point. It is about 4½

miles long north and south, and its greatest width is about 2 miles. Its southern end, which rises abruptly from a depth of 8 fathoms to 10 feet, is about ½ mile northward of Handkerchief Shoal Light vessel and 5¼ miles SW ¾ W from Monomoy Point Lighthouse. Its northern end rising gradually from 3½ fathoms to 15 feet, lies about 3 miles WNW ½ W from Monomoy Point Lighthouse.'

"The shoals are undoubtedly of a shifting character. A comparison of Coast Survey charts issued from 1860 to the present time shows enormous changes in the channels and in the shape and position of the various shoals.

"On the chart of 1860 the principal passage from Butlers Hole (deep water southwest of Shovelful Shoal Light Vessel) to the ocean was due east from Pollock Rip Light Vessel through a 5-fathom passage south of the broken part of Pollock Rip. The chart of 1874 shows this passage closed by the 5-fathom contour, which is continuous from off Chatham around the entire group of the Monomoy Shoals, the distance between the outside and inside 5-fathom curves being but 600 yards, with a depth of 4¾ fathoms between. The 1885 chart shows this distance to be about 800 yards, with 3¼ fathoms between and with several small shoals carrying less than 3 fathoms in the immediate vicinity. The 1888 chart shows this distance to be about 2 500 yards, with a minimum depth of 3¼ fathoms. The 1894 and 1900 charts give the distance as about 900 yards, with a minimum depth of 3¼ fathoms. The 1908 chart gives the extreme distance between the inside and outside 5-fathom contours as about 3 600 yards, with a minimum depth of 3½ fathoms, but with an intervening hole of 5 fathoms. The position of this easterly passage moved south from its 1860 position, the course from the Pollock Rip Light Vessel changing from due east to about southeast.

"The Broken Part of Pollock Rip has recently made out about 1 200 feet to the westward, considerably narrowing the northern entrance.

"In 1860 the Shovelful Shoal and Bearse Shoal, as defined by the 18-foot contour, were continuous and separated from Pollock Rip Shoal. In 1874 the first two of these were separated and the last two were joined together, with the southern part of Pollock Rip Shoal broken into a number of smaller shoals, which condition has continued up to the latest chart, but with varying outlines on the successive charts. The Handkerchief Shoal, which is rather more protected from the heaviest waves than the outlying shoals and therefore more nearly continuous in form, had approximately the following areas inclosed with the 18-foot curve (the dates refer to the dates of issue of charts):

"1860	1 900 acres.
1888	2 660 "
1894	2 980 "
1908	3 230 "

"The above shows a continuous increase amounting to 70% in 48 years.

"The area of water exceeding 5 fathoms in depth in the eastern extension of Butlers Hole, within which area are stationed the Shovelful Shoal and Pollock Rip Light Vessels, and limited on the west by a line drawn from the northern limit of Stone Horse Shoal to Monomoy Point, is as follows:

"1860	3 200 acres.
1888	3 000 "
1894	3 600 "
1900	3 700 "
1908	2 670 " "

The foregoing quotations show clearly the character of the shoals, how they change in position from time to time, and how, in certain areas, there is a steady accretion. The difficulties of navigating the tortuous channels, even well-lighted and marked as they are by frequent buoys and light vessels, are greatly increased by the frequently occurring dense fogs. These fogs are caused by the condensation following the contact of the warm easterly current with the colder current setting down from the coast of Maine. The Pollock Rip Light Vessel reports an average of 1 100 hours of fog occurring on 130 days per annum. So persistent and so thick are these fogs that vessels are held for days at a time at Provincetown or Vineyard Haven, unwilling to venture the passage across the shoals. The dangers are still further increased by the low-lying coast of the Cape, which becomes an exposed lee shore during east and northeast gales.

It is estimated that 22 000 000 tons of freight are carried annually around the Cape, a volume of coastwise traffic that greatly exceeds any other section of the American seaboard. It is not surprising, therefore, that the waters between Martha's Vineyard and Cape Cod Light claim the greatest toll in men, vessels, and cargo.

Fig. 1, taken from the most recent United States Coast Survey charts, shows the contour of Cape Cod, the adjacent islands, and Nantucket Shoals.

Water-borne traffic going around the Cape has the choice of two routes, either completely avoiding the shoals by passing outside of Nantucket Light Vessel, or by passing through Vineyard Sound and crossing the shoals, either through Pollock Rip, the usual course, or

south of the Great Round Shoal. Between New York and Boston the distances by these routes are:

Nantucket Light Vessel	408 miles.
Great Round Shoal	350 "
Pollock Rip	342 "

By going through Hell Gate and Long Island Sound, the first distance can be reduced by 6 miles and the last two by 16 miles. The courses are shown on Fig. 1. The distance between New York and Boston *via* Hell Gate, Long Island Sound, and the Cape Cod Canal, is 264 miles.

To avoid the shoals and fogs, with their dangers and delays, projects for a trade route *via* Buzzards Bay and a canal connecting it with Cape Cod Bay have been proposed for nearly 300 years; in fact, a canal across the neck of Cape Cod has been longer under consideration than any other public work in the United States.

The first use of this route for commercial purposes was made by Miles Standish in September, 1623, when he ascended the Scusset River, a small stream that flowed into Cape Cod Bay about 20 miles south of the Pilgrim settlement at Plymouth, and, after crossing the narrow intervening low ridge of land, met the vessels of the Dutch traders from New Amsterdam, under the command of Isaac de Resieres, who had ascended the Manomet (since corrupted into Monument) River from Buzzards Bay, laden chiefly with provisions to relieve the pressing needs of the Plymouth settlers. From this beginning there was immediately established a regular traffic between the Dutch and English colonists.

As the land separating the rivers, which could be easily ascended by the small boats then in use, was only 3 miles wide, and as its elevation was less than 30 ft. above high water, it was but natural that it was soon suggested to make a through route and eliminate the portage by digging a canal.

There is a record in the quaint diary of one Samuel Sewall, under date of October 26th, 1676, that "Mr. Smith of Sandwich rode with me and showed me the place which some had thought to cut for to make a passage from the south sea to the north." In 1697 the project received official recognition, as the General Court adopted this resolve:

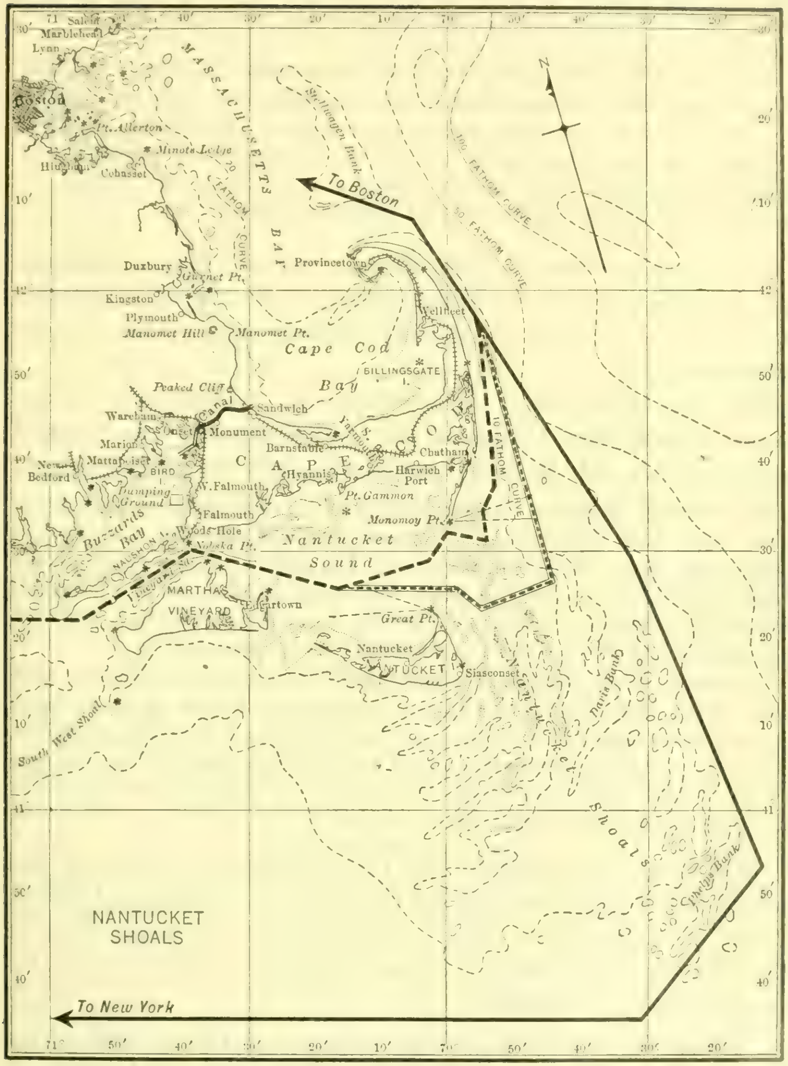

Boston
Lynn
Marblehead
MASSACHUSETTS BAY
Cape Cod Bay
Nantucket Sound
Buzzards Bay
To Boston
To New York
NANTUCKET SHOALS
Provincetown
Wellfleet
Chatham
Plymouth
Kingston
Duxbury
Sandwich
Monument
Barnstable
Hyannis
Falmouth
Woods Hole
New Bedford
Marion
Wareham
Onset
Canal
MARTHAS VINEYARD
Edgartown
Nantucket
Siasconset
Great Pt.
Monomoy Pt.
Pt. Gammon
Harwich Port
Manomet Pt.
Peaked Cliff
BILLINGSGATE
Stellwagen Bank
100 FATHOM CURVE
50 FATHOM CURVE
10 FATHOM CURVE
20 FATHOM CURVE
Davis Bank
Phelps Bank
South West Shoal
Minots Ledge
Cohasset
Hingham
Pt. Allerton
N

FIG. 1.

Whereas, It is thought by many to be very necessary for the preservation of man and estates, and very profitable and useful to the public, if a passage be cut through the land at Sandwich from Barnstable Bay, so called, into Monament Bay, for vessels to pass to and from the western part of this country,

Ordered, That Mr. John Otis, of Barnstable, Captain William Bassett, and Mr. Thomas Smith, of Sandwich, be and are hereby appointed to view the place, and make report to this Court, at their next sessions, what they judge will be the General Conveniences and inconveniences that may accrue thereby, and what the charge of the same may be, and probability of effecting thereof.

It is probable that the "Mr. Thomas Smith" of the Committee was the same person who acted as guide to Mr. Sewall. Unfortunately, there is no record of the report made by Messrs. Otis, Bassett, and Smith.

The next official action by the Colony of Massachusetts did not take place until May, 1776, when the General Court, as the Colonial Legislature was and the State Legislature is still described, resolved:

In Council, *Whereas,* It is represented to this Court that a navigable canal may without much difficulty be cut through the isthmus which separates Buzzards Bay and Barnstable Bay, whereby the Hazardous Navigation round Cape Cod, both on account of the shoals and enemy, may be prevented, and a safe communication between this colony and the southern colonies be so far secured,

Resolved, That James Bowdoin and William Sever, Esqrs., with such as the Hon. House shall join, or the major part of them, be a committee to repair to the town of Sandwich, and view the premises, and report whether the cutting of a canal as aforesaid be practicable or not. And they are hereby authorized to employ any necessary surveyors and assistants for that purpose.

This Committee appointed Mr. Thomas Machin as Engineer, and undertook to prepare, probably for the first time, a survey and definite plans. Mr. Machin had scarcely entered upon his labors when he was called to other duty by George Washington, who wrote to the Chairman of the Committee:

"The great demand we have for engineers in this department has obliged me to order Mr. Machin hither to assist in that branch of the business."

After the Revolution, in 1791, the Commonwealth of Massachusetts appointed another committee to examine and report. This Com-

mittee employed James Winthrop and John Hills to make surveys and plans, and these engineers laid out a canal, practically on the route of the present one, using the Monument River, 4 rods wide, with three sets of double locks, 30 ft. wide and 120 ft. long, and at an estimated cost of £70 707/10/00, including protecting piers in Cape Cod Bay. From this date until 1818 the Legislature had the project under continuous consideration.

In 1818 Col. Loammi Baldwin, the Engineer of the Union Canal Company of Pennsylvania, was retained by some capitalists of Boston, including Israel Thorndike and Thomas H. Perkins, to study the question. Baldwin's plan, which was the most complete that had been produced, avoided the Monument River, and used the Back River as the outlet to Buzzards Bay.

In 1808, Albert Gallatin, Secretary of the Treasury, directed attention to the canal as of strategic use in time of war, a necessity that was appreciated shortly afterward in the war of 1812, as British cruisers maintained a trying blockade along the coast between Sandy Hook and Boston Harbor. In 1818 the Senate requested the President to order a survey. Nothing, however, was done until 1824, when Congress passed an act directing that full surveys be made. These surveys were in the charge of Maj. P. H. Perault, U. S. Topographical Engineer, who reported in 1825—a report which was further examined and approved by the Board of Internal Improvements for the State of Massachusetts. This report was ordered printed by Congress in 1830.

Of course, all previous plans described a canal of very small dimensions, and the canal projected by the United States was undoubtedly considerably larger than any that had been contemplated before. Even this canal, however, according to modern standards, was quite a small affair. It was to have a bottom width of 36 ft., a water surface of 60 ft., and a depth of 8 ft. Locks were planned.

Four different arrangements of locks and levels were considered, but the one finally recommended for adoption was a single level, 8 miles 524 yd. long, with the bottom on the plane of low tide in Barnstable (Cape Cod) Bay, with a tidal lock at each end. These locks were to have a length of 107 ft. and a width of 26 ft. The cost was estimated at $669 522, in which bridges were put down at $2 000. The route selected was the same as that proposed by Col. Baldwin, using Back River. Water to supply the summit level was to be furnished by

the rise in tide at the Cape Cod Bay end, admitted through regulating sluices. The Board felt that Herring Pond and its tributaries would not supply sufficient water to permit thirty-six passages per day, the estimated possible number.

With locks, the variant in the route from the Monument to Back River, as proposed by Col. Baldwin, and the Board of Internal Improvement, is feasible, but the resulting advantage and economy are not apparent.

At the time it was confidently expected that the work would be undertaken at once. When Maj. Guillaume Tell Poussin, the eminent French engineer, at one time an officer in the American Army, wrote his celebrated report on "Travaux d'Améliorations Intérieures Projetés ou Exécutés par le Gouvernement Général des États Unis", published in 1834 after his visit to this country in 1831, he described the canal as one of the great pieces of public work about to be undertaken.

The project then lay dormant for 30 years, until 1860, when it was revived by the Governor of Massachusetts calling attention to it in his annual message. The Legislature appointed a committee which reported in favor of employing engineers to restudy the matter thoroughly, as a canal seemed both feasible and desirable. The Legislature adopted the suggestion, and appointed such a committee with powers, and again another committee in 1861, which committees united in a report in November, 1862, printed as Public Document No. 41, in 1864. This report is of great value, as it reviews the whole history of the enterprise and gives many statistics of traffic and other matters having a bearing.

As to surveys and plans, the Committee availed itself first of a suggestion of Professor A. D. Bache, Superintendent of the United States Coast Survey, to make use of officers of that service. The Commissioners for Boston Harbor, the late Joseph G. Totten, Hon. M. Am. Soc. C. E., Brigadier-General, U. S. Topographical Engineers, the late A. D. Bache, Hon. M. Am. Soc. C. E., U. S. Coast Survey, and Commander C. H. Davis, U. S. Navy, Superintendent of the Naval Academy, made a report to the Committee, giving recommendations as to locks and breakwaters in Cape Cod Bay and a most important analysis of the tidal conditions, based on observations made by the late Henry Mitchell, M. Am. Soc. C. E., then an assistant in the Coast Survey.

On receipt of this preliminary report the Committee engaged Mr. George R. Baldwin to complete the surveys and make plans. Mr. Baldwin accepted as his location the Back River route, following Col. Loammi Baldwin and the Board of Internal Improvement, but the cross-section was much greater than anything hitherto proposed, having a bottom width of no less than 120 ft. and a depth of 18 ft. Two locks were contemplated, one at each end, with two chambers in tandem, 200 and 132 ft. long, or with a combined usable length of 350 ft., 96 ft. wide. The estimated cost varied from $9 558 000 to $9 915 000, according to variations in details, but including the locks, changing the railway, and three large breakwaters.

The Committee found that about 10 000 vessels passed around the Cape each year, carrying miscellaneous cargo, of which coal contributed 370 827 tons in 1859. Between 1834 and 1859, both years inclusive, there had been 827 marine disasters on the Cape, involving 4 steamers, 40 ships, 71 barks, 191 brigs, 492 schooners, and 29 sloops, the average annual value of the loss being nearly $600 000. The Committee estimated that the annual saving to navigation resulting from the construction of the canal was $1 543 375, on the basis of 45% of the traffic using it.

The interesting feature of the report, however, was the first recorded appreciation of a canal without locks, and apparently this suggestion came from the Committee and not from any of its professional advisers, all of whom in their reports discussed locks only. The Committee stated:

"The peculiarities attending the operation of the two tide waves upon the coast has in every instance suggested to the engineers the necessity of using locks for this canal. * * *

"If some plan could be devised to overcome the force of the currents, and thereby form a free channel for the transit of vessels through from bay to bay, there could hardly be a difference of opinion upon the propriety of constructing the proposed passage, and the question whether the currents can be controlled in any way than by locks, deserves some consideration. This question was not particularly examined while the U. S. Commissioners were connected with the surveys and soundings, and their public engagements since that time have deprived the Committee of their judgment and advice upon this branch of the subject.

"* * * It may not be unreasonable to suppose that some method of avoiding the force of the currents might be discovered without the cost and delay of using locks and gates."

This vague suggestion of the Legislative Committee was crystallized into form in 1870 by Brevet Maj.-Gen. J. G. Foster, Lt.-Col. of Engineers, U. S. A., who pointed out, in a report to the Chief of Engineers, that although there was a considerable and varying difference in head at the ends of the canal, nevertheless the resulting current would not be sufficient to require locks. This contribution of General Foster's changed completely the whole character of the enterprise, because, after the publication of his report, a canal with locks was not again considered. He recommended a canal with dimensions much greater than had been previously contemplated, with a bottom width of 198 ft. and a depth of 23 ft. at mean low water.

Gen. Foster's report performed another service. It showed the great volume of traffic passing around the Cape, which could be accommodated and accommodated only by a waterway free from artificial obstructions, and, appearing as it did at the period following the war between the States, when all projects for increased transportation facilities were being eagerly taken up, it directed the attention of capitalists and promoters to the possibilities of the canal. Although no further efforts were made by either the Federal or State Governments to construct the canal, efforts by groups of financiers under private charters were continuous from 1870, the date of Gen. Foster's report, to the actual construction of the canal as described in this paper.

In 1870 a charter was given by the State to the Cape Cod Ship Canal Company, among whose incorporators were Alpheus Hardy, Thomas Russell, Charles H. Allen, Rufus Ingalls, and Charles A. Secor. This charter was regarded with favor by committees of Congress and the Legislature of the State, but nothing was accomplished, and at length it was allowed to lapse, after it had been extended by the Legislature several times. The company referred the tidal questions to Clemens Herschel, Past-President, Am. Soc. C. E., who confirmed the previously expressed opinion of Gen. Foster that the resulting current in a sea-level canal would not be sufficiently swift to prevent passage, and that a lock was not only unnecessary, but detrimental.

Although Massachusetts at an early date provided for the construction of railroads by general legislation, thus doing away with special legislative charters, no such provision was made in the general

laws concerning the construction of canals, so that recourse to the Legislature for powers has always been necessary.

When the charter to the Cape Cod Ship Canal Company lapsed in 1880, a new one was granted to Henry M. Whitney, William C. Whitney, Henry F. Dimock, Charles T. Barney, Holcomb Hosford, and associates, for the Cape Cod Canal Company. These gentlemen investigated the subject, retaining George S. Greene, Jr., M. Am. Soc. C. E., as Consulting Engineer, but they, too, permitted their charter to lapse.

In 1883 another act was passed by the Legislature, incorporating the Cape Cod Ship Canal Company, with a capital stock of \$5 000 000, the persons named being William Seward, Jr., George S. Hall, Samuel Fessenden, Edwin Reed, William A. Clark, Jr., Joseph T. Hoile, Walter Lawton, William F. Drake, and William Parker. This company made a contract with Frederic A. Lockwood to construct the canal. Mr. Lockwood was a singular genius. At one time he was a Baptist minister, but, being of a mechanical turn, he established a machine shop in East Boston, and there designed a curious type of suction dredge, under the patents of one John A. Ball of California. Being a man of much force and power of persuasion, he organized and procured the charter for the company just mentioned, primarily in order to give work to his machine shop and create an opportunity to use his dredge, he having acquired from Mr. Whitney and his associates their plans and surveys and whatever rights they possessed. He appointed Mr. George H. Titcomb Chief Engineer and Mr. Charles M. Thompson Assistant Engineer. The latter gentleman remained at Sandwich even after the Lockwood efforts came to an end, assisting the subsequent companies, and became Real Estate Agent of the present company, which position he held until his death in March, 1914.

Now, for the first time since Miles Standish began the trading route in 1623, and after all the fruitless surveys and plans by the United States, by the Commonwealth of Massachusetts, and various private parties, actual work was begun. Lockwood built his dredge, and with it cut through the open beach just north of Sandwich and near the mouth of the Scusset River. In order to obtain the necessary funds to carry on the work, Lockwood succeeded in persuading Mr. Quincy A. Shaw, of Boston, to advance them. With this aid, Lock-

wood and his singular excavating machine made a channel nearly a mile long, about 15 ft. deep and perhaps 100 ft. wide through the sandy marshes of the Scusset. While thus at work, Lockwood suffered a stroke of apoplexy, completely disabling him. He then conveyed to Col. Thomas L. Livermore, of Boston, as Trustee, all title in the chartered company, canal, land, and dredge, as security for Mr. Shaw's advances and some minor obligations. With further capital advanced by Mr. Shaw, the Trustee continued dredging for several months, carrying the excavation of the canal to a total of about 700 000 cu. yd., and acquiring title in fee to land which, with that purchased by Mr. Lockwood, amounted to about 1 000 acres. Then, probably realizing the hopelessness of completing the work with a single dredge, and especially with such a dredge as the one in hand, the Trustee stopped work, and the dredge was subsequently and mischievously set on fire and completely destroyed. The action of the waves and littoral drift closed the entrance through the beach and filled perhaps one-half of the excavation with sand.

If little physical result was accomplished, the Lockwood attempt is entitled to the honor of the first actual construction and a demonstration that some people were at length willing to do more than make surveys, and great credit must be given to Mr. Shaw and his Trustee, Col. Livermore, for having perfected the titles to so many parcels of land, and especially for keeping them intact and free from physical encumbrances that would have prevented the construction of the canal. Other routes, and variations of the route adopted by Lockwood's company, have been considered, but the one he selected —that since adopted—is the only one that is feasible. Had the land passed back into the hands of its many original holders, its re-acquisition would have been difficult and perhaps so expensive that the cost would have been prohibitory; or had it been "improved", canal construction would have been impossible. When the present company took up the work, the fact that more than 80% of the right of way could be acquired in fee simple at a single purchase, and at an ascertained reasonable cost, contributed in no small degree to the favorable consideration of the project. Other men less far-sighted than Col. Livermore would not have had the courage to keep the holdings together, and the failure to do so certainly would have jeopardized the realization of the canal, if not actually preventing it.

Following the cessation of work under the Lockwood charter, peace reigned for a few years, broken by the passage in 1891 of a charter to the Boston, Cape Cod and New York Canal Company, under which, however, nothing was done.

In 1893 the Legislature acted again, this time in granting a charter to the "Old Colony and Interior Canal Company", among whose incorporators were two prominent contractors, James D. Leary and Warren Roosevelt, and an energetic attorney of New York, William G. Bussey, of whom more later. This charter, which for the sake of safety repealed in terms all prior charters, provided for a choice of routes *via* the Monument or Bass Rivers, the latter flowing into Vineyard Sound. The Bass River route would have given a shorter canal, but with a less saving in distance, and with a failure to avoid fogs and very bad tidal currents.

These gentlemen did nothing with their grant, as likewise Oliver Ames, of Boston, and associates with a charter to the Massachusetts Ship Canal Company passed in 1895.

The Massachusetts Maritime Canal Company was the next step, chartered in June, 1896, the projector being Mr. William G. Bussey, and one of the incorporators being the late Elmer L. Corthell, Past-President, Am. Soc. C. E. The charter called for a canal of increased dimensions, *viz.,* a depth of 25 ft. at mean low water and a bottom width of 100 ft. Full and complete engineering and commercial investigations were made by Mr. (later Dr.) Corthell and the late Alfred P. Boller, M. Am. Soc. C. E. Mr. Bussey, ably assisted by Mr. Corthell, made every effort to secure the necessary capital, interesting in the project men like Myron T. Herrick of Cleveland and Lewis Nixon of New York, but finally they were obliged to let the charter lapse.

The next, and as it proved the last, step, was an application by Mr. DeWitt C. Flanagan, in 1899, for a charter, which was passed on June 1st of that year, incorporating the Boston, Cape Cod and New York Canal Company. The incorporators named in the act are Alexander Dow, David W. Belding, Charles E. Hoge, Richard G. Peters, Thomas F. McGarry, Walter Clifford, Charles H. Phelps, DeWitt C. Flanagan, and William O. Brown. Mr. C. C. Dodge was elected President of the company, holding the office until his death in 1910.

The company appointed Dr. Corthell its Engineer with Mr. Charles M. Thompson in charge on the ground. Arrangements were made with the Maryland Trust Company of Baltimore to assist in the financing and that company appointed the late Alfred L. Rives, M. Am. Soc. C. E., Colonel, U. S. Engrs., as engineer in its behalf. Col. Rives, a member of the Virginia family of that name, and formerly an officer in the Confederate Army, had been for some years Superintendent of the Panama Railroad for the French company that owned the railroad and was building the Panama Canal. Dr. Corthell and Col. Rives acted as joint engineering advisers, rendering valuable assistance in the preparation of plans and in various hearings before the State authorities. Then Mr. Bussey re-appeared on the scene and, as counsel, prepared an operating plan for carrying out the work. Unfortunately, however, unfavorable financial conditions arose, the Maryland Trust Company became involved, Mr. Bussey and Col. Rives died, and Mr. Flanagan, in order to prevent the charter from lapsing, was compelled to use his own private means to make the deposit of $200 000 with the State Treasurer and $25 000 with the County Treasurer, as required by the charter, to insure payment for land expropriated and claims for damages.

Finally, in 1904, Mr. Flanagan laid the project before Messrs. August Belmont and Company of New York, who promised to take it up when the general financial outlook should brighten.

Before considering the charter and describing the canal, it should be remarked that though there is only one feasible route for a canal across Cape Cod, as stated before, other routes for a canal westward from Massachusetts Bay, of quite different character and location, have been proposed. Of these, the one most persistently advocated was a canal from Fore River, near Boston, *via* Brockton, Bridgewater, and Taunton to Narragansett Bay. Such a canal and the others on similar inland routes were barge and not ship canals. They necessarily called for large investment and required many locks, and as the water in the central portion would have been fresh, such canals would have been inoperative—like the Erie Canal—during the winter. Full plans and estimates of these canals have never been prepared.

The act incorporating the Boston, New York and Cape Cod Canal Company is known as Chapter 448, Acts of 1899, and has been amended by Chapter 476 of the Acts of 1900 and Chapter 519 of the Laws

of 1910. These three acts constitute the charter of the Canal Company, from which it derives all its powers and rights. This charter provides:

1.—That the company may issue its capital stock to the extent of not exceeding \$6 000 000 and bonds to the extent of \$6 000 000;

2.—Right to construct and operate a canal from Cape Cod Bay to Buzzards Bay, with all structures, wharves, docks, breakwaters, etc., convenient for the canal, and operate steam and other vessels;

3.—That the canal, including its approaches in the open waters of Cape Cod and Buzzards Bays, shall have a minimum depth at mean low water of 25 ft., a minimum width on the bottom of 100 ft. at that depth, side slopes of not steeper than 2 horizontal to 1 vertical, and consequently a minimum water surface of 200 ft.;

4.—Powers for taking land and liability for damages similar to those of railroads;

5.—Obligation to reconstruct the portion of the Old Colony Railroad (leased to the New York, New Haven and Hartford Railroad) where affected by the construction of the canal, including a bridge or tunnel across the canal;

6.—Obligation to provide and maintain, without charge, ferries, bridges, or tunnels for highways;

7.—Obligation to construct highways to connect with the crossings and to replace those destroyed by the canal;

8.—Obligation to deposit with the Treasurer of the Commonwealth \$200 000 and with the Treasurer of Barnstable County the sum of \$25 000 as guaranties that land damage claims will be paid;

9.—Power to charge tolls for the use of the canal and for towing at such rates as the directors may determine;

10.—Punishment for wilful damage of the canal by payment to the company of treble the amount of damage sustained and by a fine not exceeding \$1 000 or imprisonment for not exceeding one year;

11.—Official control by the:

Harbor and Land Commission as to approval of general plans;

Railroad Commission (now the Public Service Commission) as to:

Relocation of Old Colony Railroad;

Acceptance of bridge or tunnel for crossing of the railroad over or under the canal;

Fixing of rules for operating said bridge;

Joint Board, composed of the above two boards sitting as a single board, as to:

Issue of capital;

Point of crossing the canal by the Old Colony Railroad;

Method of crossing the canal by highways, whether bridge, tunnel, or ferry;

General supervision of the work;

County Commissioners of Barnstable County and Selectmen of the towns passed through, as to:

Relocation of highways;

Points of highway crossing of the canal;

Condemnation of property.

The charter also provided that the Harbor and Land Commissioners, the Railroad Commissioners, or the Joint Board may employ an engineer or engineers whose compensation shall be paid by the Canal Company.

In 1909 Mr. Belmont decided to begin construction, and, in order to provide the necessary legal machinery through which financial arrangements could be made, he organized the Cape Cod Construction Company, which, with the consent of the Railroad Commission of Massachusetts, took the contract to construct the canal for the amount of the bonds and stock authorized by the charter, namely, bonds bearing interest at 5% to the par value of $6 000 000 and 59 900 shares of stock with a par value of $100 each, 100 shares of stock having been previously authorized and issued to the incorporators.

The directors and officers of the Cape Cod Construction Company, since the work began, have been August Belmont, Charles H. Allen, F. R. Appleton, E. Mora Davison, A. L. Devens, DeWitt C. Flanagan, W. A. Harriman, E. W. Lancaster, L. F. Loree, Jacob W. Miller, William Barclay Parsons, F. DeC. Sullivan, Frederick D. Underwood, and H. P. Wilson. The executive officers have been Mr. Belmont, President; Messrs. Miller and Devens, Vice-Presidents; Mr. John J. Coakley, Treasurer, and Mr. U. A. Murdock, Secretary.

On the completion of the canal (when the contract between the Canal Company and the Construction Company was declared completed), the above gentlemen, except Mr. Devens, who died while the work was in progress, and Mr. Davison, became directors of the Boston, Cape Cod and New York Canal Company in charge of

operation, with Mr. Belmont as President, and Mr. Miller as Vice-President and General Manager.

Prior to action by Mr. Belmont, Mr. Flanagan and his associates had had surveys made and plans prepared under the direction of Dr. Corthell and Col. Rives. When the Cape Cod Construction Company was formed, the writer was appointed Chief Engineer of that company and of the Canal Company as well, and made a new survey of the route. The location finally adopted was substantially that made by Messrs. Corthell and Rives, except that, in actual construction, the center line of the canal was placed generally 50 ft. north of and parallel to the center line of the right of way, so that a canal with a bottom width of 200 ft. could be constructed by widening on one side only for the most part. This additional construction would place the axis of the canal coincident with that of the right of way. Certain modifications in the location of the approach channel in Buzzards Bay from the original Corthell-Rives plans were also made as the result of further study subsequent to their surveys.

The location as thus determined commenced in the open waters of Cape Cod or Barnstable Bay, off the unbroken sand beach, about 3 miles north of the Village of Sandwich and 20 miles south of the City of Plymouth. It then traversed the low-lying marshes of the Scusset River and crossed the land forming the divide between Cape Cod and Buzzards Bays, passing through a depression at the Village of Bournedale, where the surface of the ground on the location had an elevation of only 30 ft. above mean sea level, although it rose, after a level width of about 1 000 ft., occupied by the location, to an elevation of about 125 ft. on both sides. Then the location followed the general line of the Monument River, the depth of which varied from 1 to 4 ft., to Buzzards Bay. In Buzzards Bay the existence of shallow shoals with many boulders prevented (except at prohibitive cost) the construction of a straight channel from the mouth of the Monument River to deep water in Buzzard's Bay off Wings Neck Light, so the location as adopted followed the general line of the existing natural channel, which required two turns.

The details of this location, with a profile of the surface along the center line of the canal, are shown by Fig. 2. A line drawn from the center of the canal at the mouth of the Monument River

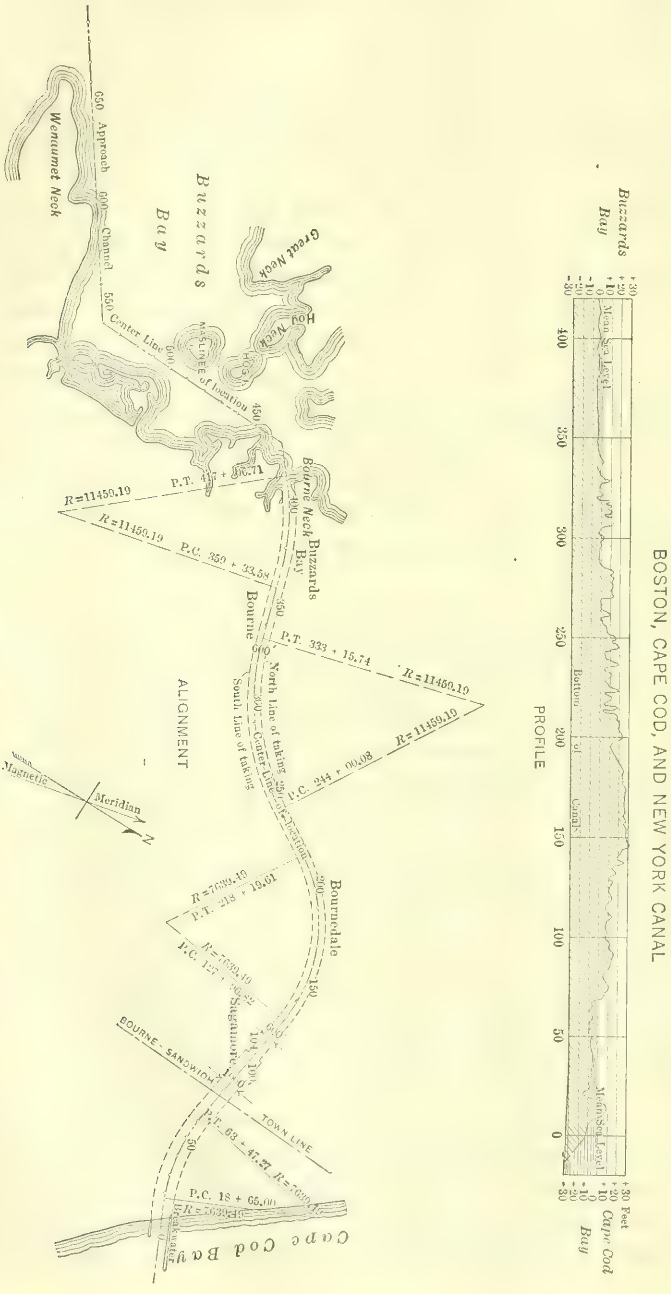

BOSTON, CAPE COD, AND NEW YORK CANAL
PROFILE
ALIGNMENT
Buzzards Bay
Cape Cod Bay
Wenaumet Neck
Great Neck
Hog Neck
Bourne Neck
Bourne
Bournedale
Sagamore
Approach Channel
Center Line
Center Line of location
North Line of taking
South Line of taking
BOURNE - SANDWICH TOWN LINE
Breakwater
Magnetic Meridian
R=11459.19
P.T. 333 + 15.74
P.C. 244 + 00.08
P.C. 359 + 33.58
R=7639.49
P.T. 218 + 19.61
P.T. 63 + 47.27
P.C. 18 + 65.00
Mean Sea Level
Bottom of Canal
+30 Feet

FIG. 2.

to the center of the canal at the Cape Cod Bay end lies almost exactly due east and west (magnetic). The canal, therefore, was considered as running east and west, and for convenience this was considered diagrammatically true of the Buzzards Bay approach, although the actual variations are considerable.

The portion of the canal under the jurisdiction of, and control by, the State, and covered by the charter, extends from the beach line of Cape Cod Bay, known as Station 11 + 70, to the mouth of the Monument River, known as Station 417 + 18.4, both being stations of the canal survey. For such distance, 40 548.40 ft. the company owns a right of way 1 000 ft. wide for 9 230 ft., at the east end, and 600 ft. wide from Station 104 + 00 to Station 417 + 18.4. This length is referred to as the "canal proper", being the limits of the portion belonging to the company and over which tolls can be collected. The approaches in Cape Cod Bay and in Buzzards Bay beyond these limits are in open waters, over which the United States maintains jurisdiction and for which there is no right of way or charter, excavation having been done under a permit from the War Department.

The original plans of Messrs. Corthell and Rives contemplated three passing places, with an extra bottom width of 100 ft., which, after discussion with the engineer of the Joint Board, were concentrated at the ends, so that the canal as built, out of a total length of 68 600 ft. has only 30 800 ft., or less than 6 miles out of 13 miles, with the minimum width of 100 ft. on the bottom. In the canal proper, changes in direction are effected by curves with radii varying from 7 589.49 ft. (1.5 miles nearly) to 15 113.04 ft. (2.86 miles), the longest curve being 9 522 ft.; and in Buzzards Bay by the tangents meeting with deflection angles without curves, a straight-line widening being made on the inside of the angles to give vessels swinging room in turning.

The chief features of the location are as follows:

	Length: In feet.	In miles.
Canal proper (Stations 11 + 70 to 417 + 18.4)	40 548.40 =	7.68
Dredged approach, Cape Cod Bay (Stations 0 — 14 to 11 + 70)	2 570.00 =	0.49
Dredged approach, Buzzards Bay (Stations 417 + 18.4 to 672)	25 481.60 =	4.83

	Length: In feet.	In miles.
Total dredged waterway (Stations 0 — 14 to 672)	68 600.00 =	13.00
Tangents in canal proper	12 342.66 =	2.34
Tangents in Cape Cod Bay	2 570.00 =	0.49
Tangents in Buzzards Bay	25 493.29 =	4.83
Number of curves in canal proper		4

Length of curves in canal proper:

Radius, in feet.	Length, in feet.
{ 3 536.25	{ 7 589.49 to 7 639.49
{ 1 429.81	{ 11 509.19
9 009.33	7 639.49
{ 6 999.92	{ 11 409.19
{ 2 522.28	{ 15 113.04
5 773.33	11 459.19

Total length of curves in canal proper: 29 270.92 ft. = 5.54 miles.

Number of angular turns in Buzzards Bay....................2

Deflection angles of turns: Station 430.............43° 14′

" 540.............50° 29′ 41″

Length with bottom width of	300 ft.	4 400 ft.
" " " " "	250 "	27 200 "
" " " " "	200 "	3 000 "
" " " " "	150 "	1 500 "
" " " " "	100 "	30 800 "
" " " " "	250 to 450 ft. (turns)	4 000 "
" " " " "	200 to 300 " transition	1 000 "
" " " " "	150 to 250 " "	300 "
" " " " "	100 to 200 " "	400 "

Slight apparent discrepancies will be noted in some of the foregoing figures. The explanation is that the total lengths of the canal are given as measured on the center line of the location, but the detailed lengths of the curves and tangents are those of the present canal, as actually constructed, offset from the center line of the right of way. When the canal is widened and the two center lines become coincident, the summation of the lengths of curves and tangents will agree with the total lengths as given, and the varying lengths of the radii of the compound curves will disappear.

The adopted cross-sections of the canal are shown on Fig. 3, in which the steepest slope in the canal proper is put at 1 on 2 to a point 6 ft. above high-water level; in the approaches, however, where wave action is more pronounced, and no opportunity is afforded to protect the slopes against wave action, the minimum slopes were made

1 on 3. As there is considerable difference in tidal elevation at the two ends, the mean amplitude of the tide at the eastern end being nearly 10 ft., the bottom grade of the canal is on a slope in order to give a depth of 25 ft. at mean low water. Thus the elevation of the bottom of the canal at the east end is 30 ft. below mean sea level, and at the west end it is 27.5 ft. The slope of the bottom, however, is on a curved, and not a straight, line, as the heights of mean high and mean low water in the canal proper make concave and convex curves, respectively, all of which details will be referred to later.

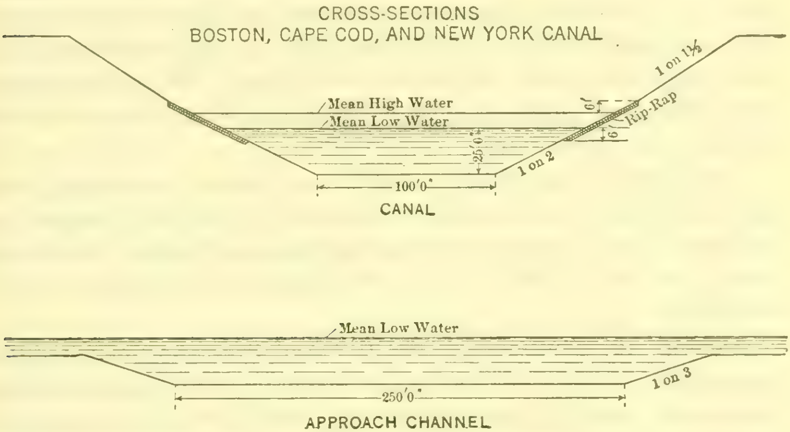


FIG. 3.

At the west end the approach and canal entrance are land locked and required no special consideration, but at the east end the canal debouches on an open sandy shore, with full exposure to winds coming from any part of the quadrant north to east. Winds west of north (magnetic) are broken by the high ground south of Plymouth, and winds east of northeast (magnetic) are partly broken by the low-lying cape 25 miles distant and by the land constantly getting nearer as the bearing approaches south, when the land makes a complete lee. Between north and northeast the winds have a clean sweep from the Maine coast, a fetch of 180 miles.

To protect the easterly entrance to the canal, and permit vessels to enter at times of storm, there were designed and constructed two parallel breakwaters, 800 ft. apart. The larger one, on the north side, has a length of 3 000 ft. from the contour of mean high water, and

provides protection against north and northeast winds. The smaller, 1 000 ft. long, on the south side, is intended to stop the littoral sand movement from the south.

Other works required were: the protection of the slopes with rip-rap from 6 ft. below mean low, to 6 ft. above mean high, water, the relocation of the railway, the construction of certain bridges and ferries, highways, lighting system, and other aids to navigation, all of which will be described in order.

The surveys and plans being complete, a contract was made on May 15th, 1909, after competitive bidding, with the Degnon Cape Cod Canal Construction Company, a contracting organization incorporated specially to excavate the canal and its approaches, build the breakwater and protect the banks with rip-rap. All other required works were specifically exempted and reserved for other contracts with this or other contractors.

This company was owned jointly by the Degnon Contracting Company of New York and the Furst-Clark Contracting Company of Baltimore, and it sub-contracted to the Degnon Company the breakwater and bank rip-rap, and to the Furst-Clark Company all the excavation.

Having thus given a condensed sketch of the history of the enterprise and of the facts leading to the execution of the main contract, the principal details of actual construction will be considered separately.

Excavation.—The work was formally begun by Mr. Belmont "turning the first sod" on June 22d, 1909, though actual construction was started on June 19th by the Degnon Contracting Company depositing the first stone in the breakwater. The contractors recognized that the breakwater must be advanced so as to afford some lee for a dredge to cut through the beach of Cape Cod Bay.

The Furst-Clark Company, dredging contractors of long and varied experience, considered that the excavation, certainly of the canal proper, could be done almost wholly by hydraulic dredges, and arranged for such plant, except in the approach channel in Buzzards Bay, where, spoil area for pumping not being available, the material had to be dug and removed in scows. They acted promptly, placing the dredge *Kennedy* at work in Buzzards Bay on August 2d. This dredge was of the "ladder" type with $\frac{5}{8}$-cu. yd. buckets; it was followed on October 25th by *Coastwise Dredge No. 1,* a small "clam-shell"

machine. The plan was that these dredges, to be assisted by others later, were to cut a deep channel through to the Monument River to permit hydraulic dredges to enter and excavate the canal proper while excavators of other types were completing the approach channel.

On October 16th the first attempt to cut through at the east end was begun by the *Mackenzie,* a 22-in. hydraulic dredge, starting as close to the beach as it could work. The attempt was not successful. As the breakwater, in its early stage, provided but little protection, and as the waters were exposed, the dredge had to be withdrawn either to Provincetown or Plymouth every time a storm, or even high wind, was threatened. Finally, in December, the plan was abandoned, and the *Nahant,* a small clam-shell dredge with a long boom, was floated in through the Scusset River and shallow channels crossing the marshes into what remained of the old Lockwood excavation. On January 24th, 1910, this dredge began to work seaward, opening the old canal which had become closed at the beach, and placing the excavated material on shore. On April 7th, it had cut a way through to open water, and on the following day the *Mackenzie* entered the canal and the *Nahant* was withdrawn.

With the idea of removing some of the over-burden in the dry, the contractors erected in December, 1909, and January, 1910, two "drag line" excavators. The former continued in operation until November, 1910, and the latter until February, 1911. The results were very unsatisfactory, as these machines were not adapted to the soil to be excavated, and were too heavy to move over rough and swampy ground. They involved the contractors in a heavy loss.

In the spring of 1910 the Furst-Clark Company placed at work another 22-in. hydraulic dredge (known as *Number 9*) at the east end to work in tandem with the *Mackenzie,* and at the west end the 4-cu. yd. dipper-dredge, *Bothfeld,* followed in August by the *Onondaga,* a 9-cu. yd. dipper-dredge, and in September by a smaller machine, the *Neponset.* In July, the *Bothfeld* having deepened the channel sufficiently, the *Warren,* a 12-in. hydraulic dredge, entered the Monument River and began spoiling on shore.

By this time the contractors realized that they faced a much more serious task than they had anticipated. The material contained so many boulders and such a high proportion of cobbles, cemented gravel, and stiff clay that it could not be excavated completely by hydraulic

dredges except in places, and recourse must be had to "dipper" machines. The contractors thereupon purchased the plant of another company, consisting of three dipper-dredges: the *National, Capitol,* and *International,* with buckets of from 5 to 6 cu. yd. and the *Federal,* a small hydraulic dredge. The first three arrived in Buzzards Bay in November and December, 1910, but the *Capitol* and *National* were at once towed around the Cape to the east end to assist the *Mackenzie,* which was having difficulty with the hard material, the *Number 9* having been already withdrawn.

At this time there were at work one first-class hydraulic dredge, one old ladder-dredge, in good condition but not adapted to this work, one small hydraulic dredge, and five dipper-dredges, all old and of designs fitted for digging only soft material, and it was evident that the plant, even as reinforced by the purchase of the hydraulic and the three dipper-dredges, would not suffice. Arrangements were then made to remove some top material by steam shovels, work which the excavators had failed to do. Two shovels, with 2 and 2½-cu. yd. buckets, were set to work during the summer of 1911, when the *Federal* started at the west end of the canal, and the *Suffolk,* another 12-in. hydraulic, took the place of the *Warren.*

The contractors were soon compelled, however, to realize that their dipper-dredges, or in fact any other dipper-dredges then in existence, were wholly unsuited to the allotted tasks, and that, if they were to complete the work, powerful modern equipment must be obtained. Then, at the end of 2 years, instead of at the outset, orders were given for the construction of two powerful dipper-dredges, the *Governor Warfield* and the *Governor Herrick.*

These dredges were designed and built by The American Locomotive Works, of Paterson, N. J., being advised by A. W. Robinson, M. Am. Soc. C. E., and Mr. A. D. Morris, Mechanical Engineer of the Furst-Clark Company. The hulls are of steel, 135 ft. long, 42 ft. wide (though 50 ft. wide over the forward spud sponsons), 7 ft. draft when resting on the spuds, and 10 ft. maximum draft when floating. The forward spuds are 70 ft. long and 42 in. square in cross-section, with "pin-up" and "pick-up" operation by cable. The stern spuds are 70 ft. long and 30 in. square, with rack and pinion pick-up. All spuds are made of braced steel plates. The forward spuds are placed so that the boom can swing through an arc of 180°, a necessary feature

when a dredge is to work in a heading and load scows alongside. The boom, 57 ft. long, is of the rigid Robinson model. The dipper-handle, of composite steel and wood construction, is 2 ft. 6 in. by 2 ft. 8 in. in cross-section, and 62 ft. 10 in. long, capable of digging in 40 ft. of water. On the boom, controlling the dipper-handle, is a "crowding" engine, which is controlled by the dipper tender.

For each operating part of the dredge there is a separate engine, as follows:

1 Main hoisting engine; two 18 by 24-in. cylinders; single condensing.
1 Backing drum, operated by main engine.
1 Swinging engine; two 10 by 14-in. cylinders; single condensing.
2 Forward spud engines; two 12 by 15-in. cylinders; single condensing.
2 Aft spud engines; two 8 by 8-in. cylinders; single condensing.
1 Crowding engine; two 12 by 15-in. cylinders; non-condensing.
1 Dipper latch engine; 7 by 24-in. cylinder; non-condensing.
2 Deck engines; two 9½ by 12-in. cylinders; single condensing.
2 Capstan engines; two 6 by 8-in. cylinders; single condensing.
1 Rock hoist engine; two 8 by 12-in. cylinders; single condensing; capable of lifting 60 tons.
1 Surface condenser; 1 500 sq. ft. cooling surface.
1 Refrigerating plant.
1 Electric light engine; 10 kw.
1 Air compressor.
5 Pumps.
2 Scotch marine boilers, 116 in. in diameter, 7 ft. long; 1 350 sq. ft. of heating surface, capable of generating steam at 135 lb. pressure.
1 Auxiliary vertical boiler; 48 in. in diameter, 9 ft. high.

The dippers are of two sizes, 10 cu. yd. for sand, gravel, etc., and 8 cu. yd. for rock. The main hoisting cable is a single line, 2¼ in. in diameter. The backing spud and swinging cables are 1¾ in. and 2¼ in. in diameter, respectively. By avoiding sharp turns in the cables and using large sheaves where turns were necessary, a very high average life of cable is secured. The main and backing cables will dig about 300 000 cu. yd., scow measure, of hard material. The spud cables will last for about 1 year of continuous work for the "pin-up" and nearly 2 years for the "pick-up" cables; the swinging cables will last from 5 to 7 months.

The vessels are equipped with quarters for double crews, including an inspector's room, and a large galley with separate messrooms for officers and crew.

These dredges were built on the canal. The plates for the hull were sent to the site punched, and, after the hulls had been constructed and launched, the several engines, built at Paterson, were shipped in pieces and erected on board.

After tuning up and some reconstruction, these two machines have made an extraordinary record. The best single day's record was more than 8 000 cu. yd., and the best month 131 000 cu. yd., in both cases "place" measurement, although dredges are usually rated by scow measurement, which includes a swell of from 15 to 20 per cent. These records were accomplished, not in an open seaway with plenty of room, but in a narrow channel, and in a closed heading involving the turning of nearly every scow so as to load both ends. The scows were also necessarily small, usually having a capacity of from 600 to 700 cu. yd., and the material dug was very hard sand when at best, and contained boulders running up to 30 and 40 tons. The average record, as shown by 9 consecutive months when engaged in straight work, was more than 95 000 cu. yd. per month.

The operating crew for double shift—so as to give continuous service—consists of from 25 to 28 men for two shifts as follows:

1 Captain,
1 Chief Engineer,
1 Second Engineer,
2 Operators,
2 Dipper tenders,
1 Handy man, with rank of dipper tender,
2 Oilers,
6 Firemen (3 shifts),
2 Mates,
6 Deckhands,
1 Cook,
1 or 2 Messboys.

When in continuous operation, 11 tons of soft coal are burned daily. The *Herrick* began work in July, and the *Warfield* in August, 1912.

While these dredges were being built, the Chief Engineer urged the contractors to get more plant at work in the central portion. One plan that was considered to increase the plant was to place the large hydraulic dredge *Mackenzie* on railway trucks and on a special ship railway transport it over the right of way from about Station 140 to about Station 220, where a pool in the Monument River could be formed to permit it to begin work. This plan was abandoned in favor of building a new hull for the *Federal,* and erecting in it new boilers but retaining the old pump and machinery, and converting the dredge at the same time into a 15-in. machine. This was done, and the reconstructed *Federal* was set to work inland in August, 1912. By thus starting an operating unit between the headings, the same advantage was obtained as sinking a shaft in tunnel work.

As excellent and economical as were the dredges *Warfield* and *Herrick,* they were built too late to permit the work to be completed within the limits of the contract. In August, 1912, the Construction Company made a separate contract with the E. W. Foley Construction Company for a steam shovel to assist in the removal of the overburden, and in the autumn of that year took up with the Degnon Cape Cod Canal Construction Company the whole question of carrying on the work. The outcome of this was the dissolution of that company, and the making, as of November 1st, 1912, of two contracts directly with the component partners in that company who had heretofore been acting as sub-contractors. The Degnon Contracting Company undertook to complete all the stone work, under the terms and conditions of the original contract, which they carried out. The Furst-Clark Construction Company took over, under a direct contract, but under different terms from the original contract, all the excavation except such as the Construction Company might do itself or under other contracts with other parties, including the single Foley contract already in force.

When it was first fully realized that the excavation was more difficult, not only than originally, but even after considerable experience, expected, the writer urged on the general contractor the advisability of extending the operations of the steam shovels, and even to take 2 or 3 miles in the central portion of the canal proper, and not only remove the part above ground-water level, which was about Elevation 108, by shovels, as they were doing in a small way, but to

erect pumps, carry the Monument River in a flume around the work and then complete the excavation of the canal to grade and rip-rap the banks in the dry, breaking up the boulders encountered for this purpose. The contractors, however, were afraid that an excavation to a depth of 36 ft. below the level of ground-water and having a width at the bottom of 100 ft. for a distance of from 10 000 to 15 000 ft. in a soil composed chiefly of sand and gravel, would be one that could not be kept sufficiently dry, and they declined to undertake it.

When the excavation contract was remade, in December, 1912, but effective as of November 1st, the Construction Company, realizing that the Furst-Clark plant could not probably complete the work as quickly as desired, decided to undertake itself the working of steam shovels below water level, and made a new contract with the Foley Company, with profits dependent on success. In addition to its 2-cu. yd. shovel already at work, the Foley Company purchased a 2½-cu. yd. shovel, which had been working in the dry under a sub-contract with the Furst-Clark Company, which work was substantially completed.

At first a stretch of work between about Stations 235 and 255 was taken, and a steam shovel put in. This section was selected because the Monument River between these points lay in its old bed, outside of the canal excavation. It was soon found that the difficulty of maintaining a sufficiently dry excavation to a depth of at least 10 ft. below ground-water was negligible, and it was decided to extend the operations. The Foley Company arranged for a third shovel.

The Monument River flows into the canal at Station 195, coming from the north and almost at right angles to it. To the eastward of this station the ground above Elevation 108 had been removed, and as this was but little higher than the elevation of the bed of the river, a small dam was sufficient to divert its flow to the eastward, so that the only water to be encountered was ground seepage. Four natural dams were left, at Stations 208, 216, 234, and 276. It was first expected to confine operations between the dams at Stations 216 and 276, but afterward, when it was seen that the advance progress of the dredge at the east end was slower than scheduled, excavation by steam shovel was extended to the section lying between Stations 216 and 208.

The pumping plant consisted of four centrifugal pumps, with 14-in. suction and 12-in. discharge, driven by General Electric induction

motors developing 50 h. p. at 550 volts; two 5-in. centrifugal pumps, direct-connected to 550-volt induction motors; one 10-in. and one 8-in. steam pump.

At one time the whole excavation from Stations 208 to 276 (6 800 ft.) was open, and a large part with an average depth of at least 20 ft. below the level of ground-water, and the foregoing plant, which had one-half in reserve, was amply sufficient to keep the trench free of water. In fact, one of the large pumps running steadily would have sufficed.

The maximum depth made by the Foley Company was a cut at Elevation 85 (mean sea level being 100), or 23 ft. below the elevation of ground-water. Between Stations 208 and 216 the bottom of the pit was at about Elevation 95; between Stations 216 and 234 at 90; and between Stations 234 and 276 from 85 to 90. Some of the boulders encountered were broken by blasting, and the pieces were used to riprap the north bank of the excavation between Stations 220 and 265.

The Foley shovels were served by narrow-gauge locomotives and 4-cu. yd. cars.

The first of the dams to be removed by a dredge was that at Station 276, then followed the dams at Stations 208 and 216, in order, leaving that at Station 234 for the last.

The Furst-Clark Company continued using its dredging equipment, but with the addition of another old dipper-dredge, the *Weymouth*, with a 5-cu. yd. dipper, while the Foley Company operated three steam shovels by the aid of pumps under a separate contract. Thus the work continued during 1913, and by June, 1914, the only remaining excavation in sight was the Foley dam at Station 234. This was removed in July, and the formal opening, but of a canal not to full depth at all points, was on July 29th, 1914. On the following day the canal was thrown open to commercial traffic by vessels drawing not more than 15 ft.

Immediately after the opening, the Furst-Clark Company objected to continuing the work of completion with traffic having the right of way, and stopped work in September. An arrangement was then effected whereby the Construction Company undertook the work of completion, the Furst-Clark Company turning over the dredge *Warfield* and some other plant for use by the Construction Company.

In the completion of the work the Construction Company used three dredges and two corps of divers and lighters. As it was decided to maintain traffic, it was desirable that the plant used in the portion of the canal where the bottom width was 100 ft. should occupy the minimum of space. An investigation showed that much of the material in this portion could be handled by a powerful suction-dredge, and such a machine was found in *Dredge No. 3,* belonging to the Metropolitan Dredging Company, Mr. R. P. Marshall, President. This dredge had a 20-in. suction, and an engine capable of developing 1 000 h. p. It was chartered, set to work in December, 1914, and dismissed from charter on October 2d, 1915, during which time it rendered most efficient service. *Dredge A,* of the Standard Engineering Company, also a 20-in. machine, was taken under charter and used to complete the excavation at the east end. The *Warfield* worked generally in the wider portions of the canal and the approaches, although it was used in the 100-ft. section, as it was found that, with a little practice, scows could be shifted from alongside to ahead and back to permit vessels to pass.

When the *Warfield* was first used in the 100-ft. section, the dredge worked only on a fair tide, as it could not move itself and the scow against the current. As soon as the spuds were lifted, the weight of the dipper on the bottom was not sufficient to hold the dredge, and it would slip back. The suggestion was made by W. J. Douglas, M. Am. Soc. C. E., Deputy Chief Engineer of the Canal Company, to turn the dredge at slack water, taking up a new position, so that it would always be working in the direction of the current. This movement was easily performed, the crew soon becoming so expert that substantially continuous service was secured.

The great obstacle to be overcome in the completion of the excavation was the removal of the boulders, which were found in surprisingly large numbers, even when the canal was considered finished. The small lumps which were expected to flatten out by wave action were found in nearly every case to be the ends of boulders, and to remove them meant the digging up of boulders that were embedded many feet. Similar experience was had on both banks, where the existence of boulders could be detected only by divers. Other boulders were found lying along the toes of the side slopes, whither they had been apparently rolled by the dipper-dredges.

These boulders were located by sweeping and by divers, and were removed either by hoisting in slings, attached by the divers, by large steam lighters, or were broken by blasting and the pieces disposed of. The blasting was done usually by placing charges of dynamite on top of the boulder, depending on the overlying water to act as tamping. The explosive used was du Pont 75% gelatine dynamite, the charge varying from 25 to 200 lb. The best results seemed to be produced by charges of about 50 lb., even if they had to be repeated, the second and subsequent charges being inserted in the fissures made by the first.

Boulders as large as 80 tons have been thus disposed of; the total number handled by divers, either by blasting or slings, was about 700, weighing in the aggregate perhaps 3 500 tons.

A large piece of work teaches lessons of two classes: successes and mistakes. The second class is quite as important as the first, if not more so. The lessons of the first class usually need no historian, for they always speak for themselves, whereas those of the second class are too frequently buried and lost sight of. The work of excavation of this canal, the largest single item in its construction, is not without lessons of the second class.

The first of these was the failure to see that in work of this character the steam shovel is superior to the dredge. The excavation contract being taken by a company whose great experience was in dredging naturally led them to use that method of attack, overlooking the wonderfully elastic capabilities of the American steam shovel.

In open water, dredging is the only method of excavating. As many units as desired can be used, and progress is not absolutely dependent on any particular one. In the case of a canal excavated through land, unless some means are devised for attack at intermediate points, advance depends entirely on the ability of the dredges at the two headings to maintain continuous performance. In practice this is very far from realization. Any accident, any stoppage for repairs, and the whole advance at that end ceases. By the very nature of the machine, there can be no reserve. Interest, hire of plant, overhead expenses, and much labor cost continue, regardless of whether progress is or is not made. Economy, therefore, demands that progress should be as nearly continuous as possible, in order to carry this heavy overburden. In the case of the Cape Cod Canal, the irregularity in the character and composition of the soil greatly hampered successful

dredging operations. A single boulder would frequently delay a whole unit—dredge, tugs, and scows—for many hours. Even when not delayed, progress was measured by the capacity of only two machines.

By the use of steam shovels, the reverse takes place. Each unit is comparatively inexpensive, and is independent of any other. Therefore, many can be used economically, and shovels can be held in reserve to take the place of any temporarily disabled, reducing loss through idle plant to the minimum. Thus a long stretch of work can be covered with plant, instead of having it concentrated at two points. With soil so variable in character as that at Cape Cod, it is of the greatest benefit to have it exposed to sight. Boulders which turned beneath water were annoying and, through delay to plant, expensive, bothered steam shovels scarcely at all. If too large to lift, they were rolled to one side, and after the shovel had passed, were broken by blasting and the parts picked up on the next cut.

Every one admitted the feasibility of removing by shovels the material lying above ground-water level, and the general contractors began such work as soon as it was seen that rapid progress by hydraulic dredges was not to be expected. The fear was of the difficulty of successfully and permanently lowering the water level. Experience showed that there was no serious difficulty in keeping the trench unwatered. The economical programme would have been to have constructed dams at about Station 112, where there was a highway, and at Station 317, where there was a railway embankment, and to have put in a battery of larger steam shovels, with dippers of 5 cu. yd. capacity, served by standard gauge equipment. These shovels would have handled without trouble all the small boulders and fragments of the blasted ones, and the large cars would have received them without damage. Between these limits there were about 6 500 000 cu. yd., of which about 4 500 000 cu. yd. were below the level of ground-water. One-half of the canal proper (20 500 ft.) could have been thus unwatered and fully completed in the dry, with all boulders removed, slopes trimmed to even planes, floor leveled, and banks carefully protected by hand-placed rip-rap.

While this work was progressing the part east of Station 112 could have been excavated by a hydraulic dredge, the Scusset marshes affording convenient spoiling area, a few hard lumps being removed at the end by a dipper-dredge. West of Station 317 two dredges of the *Warfield* type should have been built, and should have worked in tandem,

completing the channel from Wings Neck Light. In the approach channel in Buzzards Bay there were about 3 250 000 cu. yd., and between the west end of the canal proper and Station 317 about 1 750 000 cu. yd. Of the latter perhaps one-half could have been put ashore by a light-draft hydraulic dredge, as was done in part, making a good channel for one of the larger dipper-dredges to work in to complete to Station 317. In this way there would have been used two large dipper-dredges, one large hydraulic dredge, one small hydraulic dredge, one clam-shell to cut through the east beach, as was done by the *Nahant,* and four or five large steam shovels. Such a plant, by putting practically the whole length under construction simultaneously, could have completed the canal in 3 years. Instead of ten units there were actually used no less than twenty-six.

The second lesson was the use of antiquated instead of modern plant. In Table 1 the work of fifteen dredges is shown comparatively by giving the total output of each dredge while it was on the work, the equivalent number of months worked, and the average per month. This table includes all vicissitudes of the work—time lost through stress of weather, repairs, delay in the scow service, and other causes. The average, therefore, is one of good and bad, and the time each dredge worked was sufficiently long to give a fair average of all conditions. The number of months worked is a fair reduction of the time given by months when the dredge was actually in service, including the time laid up for small repairs. Where the dredge was absent from the work for one whole month that time has not been included. The names are not given, except in the cases of the *Warfield* and the *Herrick,* as it is unnecessary to make special and invidious comparisons.

TABLE 1.—COMPARISON OF THE WORK OF DREDGES.

Dredge.	Total output, in cubic yards.	Months.	Average per month, in cubic yards.
Warfield	1 265 800	14	90 000
Herrick	1 294 800	15.5	83 600
Dipper-dredge, No. 1	583 500	14	41 700
Ladder-dredge, No. 1	1 064 400	26	40 900
Dipper-dredge, No. 2	603 800	17	37 000
Dipper-dredge, No. 3	1 197 600	39.5	30 300
Dipper-dredge, No. 4	822 200	33	24 900
Dipper-dredge, No. 5	779 300	35	22 300
Seven other small dredges	732 800	48.5	15 000

From Table 1 it will be seen that the *Warfield* did nearly two and one-quarter times as much work as the nearest dipper-dredge, and four times as much as the poorest dipper-dredge, exclusive of the small dredges the output of which has been appended with an average of 15 000 cu. yd. per month. The *Warfield* was no more expensive to operate than Dipper-dredge No. 1, as the latter machine was very costly on account of repairs. The other machines were at least two-thirds as expensive to operate as the *Warfield,* although their output was very much less. In one month the thirteen other dredges would excavate, on the average, 302 000 cu. yd., or a trifle more than three times the average output of the *Warfield.* In this respect it must be kept in mind that the hard work was saved entirely for the *Warfield* and *Herrick,* as the other dredges were incapable of handling it.

It will be seen, therefore, that had specially designed dredges been set to work at the outset, they would have more than paid for themselves during the construction of the canal.

As an interesting comparison, the two 20-in. hydraulic dredges removed 2 558 300 cu. yd. in 26 months, or an average of 100 000 cu. yd. per month, and the small hydraulic dredge *Federal* removed 613 800 in 32 months, or an average of 19 200 cu. yd. per month. The excavators removed 316 200 cu. yd. in 22½ months, an average of 14 000 cu. yd. per machine per month; and the steam shovels removed 2 077 600 cu. yd. in 89½ months, an average of 23 200 cu. yd. per shovel per month, and these were small shovels served by narrow-gauge equipment, put in as an experiment. The excavators and the shovels worked on a single shift, but all the dredges worked continuously on double shift.

Including excavation made by the Metropolitan and Standard dredges and by other dredges used directly by the Canal Company, the total excavations amounted to about 15 000 000 cu. yd.

Breakwater.—The breakwater at the east end was necessary to protect the mouth of the canal from littoral drift and to afford protection to vessels entering and leaving in time of heavy weather.

The principal clauses in the specifications describing it are as follows:

The shore end shall extend from the mean high water mark at a uniform height of 10 ft. above the beach until it intersects the bluff or dune. The beach and the sides of the dune forming the surface of

this intersection shall be rip-rapped for the distance and to the extent as directed to form a secure revetment, with stones weighing at least 100 lb.

The stone will be deposited in the structure so as to construct a breakwater of a uniform width of 25 ft. at a point 18 ft. above mean low water.

From the top to a point 12 ft. below mean low water the seaward or northeasterly slope shall be 1 on 2, and thence to the base the slope on the seaward side shall be 1 on 1.

The harbor or southeasterly side, shall be constructed throughout with a slope of 1 on 1.

The mean rise and fall of the tide is about 10 ft.

The stone must weigh at least 160 lb. per cu. ft., must be strong and durable, and not subject to disintegration by being wholly or partly submerged in sea water.

Below the level of 12 ft. below mean low water, and in the core, that is to say, not nearer than 10 ft. from the outside line of the breakwater, the stone may be of any size convenient, provided that no stone shall be of less weight than 100 lb., and that, at least 50% of the stones shall weigh not less than 1 ton each.

Within the space above defined, *i. e.*, the core below 12 ft. below mean low water, coarse gravel and sand shall be dumped into and among the stones as directed, and to the quantity ordered by the Engineer.

Below 12 ft. below mean low water and on the faces of the breakwater, *i. e.*, the outward 10 ft. of each slope, no stone shall be less than 500 lb. in weight and at least 50% shall be 2 tons or more each in weight.

Above the level of 12 ft. below low water no stones shall be used that weigh less than 3 tons each, except that where directed smaller stones may be used to fill in openings and to provide firm bearings for the larger stones, and at least 25% must weigh 6 tons each. Nor shall any stone be used in this position of which the least dimension is less than one-fourth of its greatest.

In the absence of scales or other convenient method of weighing separate stones, the judgment and decision of the Engineer as to the weight of various stones and as to the proportions of stones of various weights as specified in the above paragraphs shall be final and binding upon the Contractor.

The weight of stone deposited will be determined by water displacement, and in order to determine the correct displacements, the Contractor may be required to have the vessels accurately "weighed in" and distinctly marked, at his own expense; the "weighing in" and marking to be done under supervision of the Engineer. In "weighing in" the vessel must be kept in the same fore and aft trim

that is to be used when freighting stone. When fore and aft readings of a loaded vessel differ by more than 10% of their mean, the Contractor will be required to move enough stones to make the difference between the fore and aft readings less than 10% before the stone will be received.

As the work progressed it was decided by the Engineer to depart from the usually accepted standard of breakwater construction, as illustrated in the foregoing extracts from the specifications, and omit any attempt at compact hearting by using only such small stones as came normally mixed with the large ones, and omitting all gravel and sand. The result is a breakwater with large voids, such as naturally form between large irregular blocks. The resulting effect seems to be that any vacuum following a receding wave is impossible, the large voids giving free movement to the air. Waves on striking the breakwater are broken partly by direct shock and partly by dissipation in these large voids. No stones have been displaced on the seaward side since the final setting, except at the extreme outer end, as will be referred to presently; and, in spite of the large voids, sand does not apparently travel through the breakwater. On the shore end the beach has built itself out so as to make a curve between the face of the breakwater and the shore line, which are at right angles to each other. Where the sand is thus built up it is 10 ft. higher on the outside than on the inside of the breakwater, which at that point has a thickness of 62 ft. The sand, therefore, seems to assume a slope through the breakwater of about 1 on 6.

Fig. 4 shows a cross-section of the breakwater as originally planned. In addition to the change in specifications describing composition, one change was made in the profile during construction. Although the stones were stable at a slope of 1 on 1 on the inside face, the contractors found difficulty in placing them at that slope, so that they were permitted to flatten it from low-water level up by narrowing the top from 25 ft., as shown, to about 22 ft.

The total length of the breakwater is 3 000 ft., carrying the outer end to where the water is 35 ft. deep at mean low water, the total height of the breakwater at the end being, therefore, 53 ft. In its construction, 326 456 tons of stone were used.

Parallel with the breakwater and 800 ft. from it, a smaller breakwater has been built on the south side of the channel. This smaller

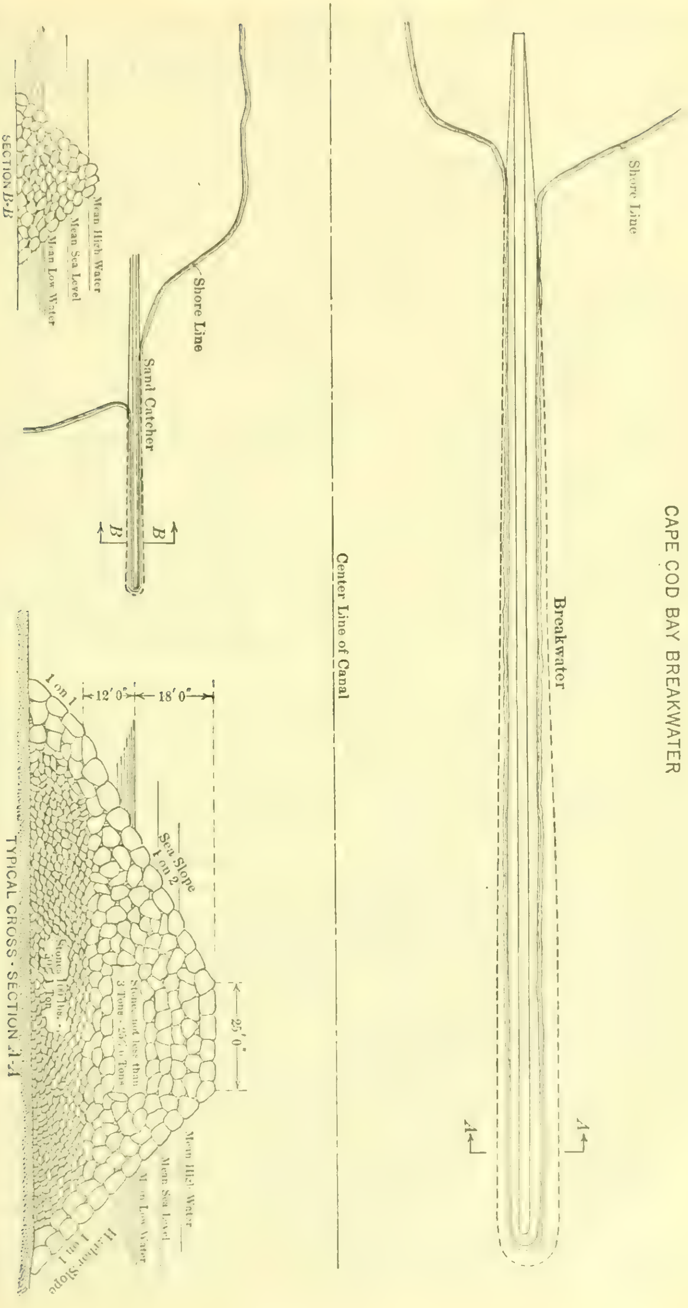

CAPE COD BAY BREAKWATER
Breakwater
Shore Line
Center Line of Canal
Shore Line
Sand Catcher
SECTION B-B
Mean High Water
Mean Sea Level
Mean Low Water
TYPICAL CROSS-SECTION A-A
Sea Slope 1 on 2
1 on 1
12' 0"
18' 0"
25' 0"
Mean High Water
Mean Sea Level
Mean Low Water
Harbor Slope 1 on 1

FIG. 4.

structure, 1 000 ft. long, and extending to a depth of 8 ft., is intended to check any littoral drift of sand from south to north being deposited in the channel. It is carried up so as to be exposed at high water, and contains 9 192 tons of stone.

The stone in the main breakwater and in the "sand catcher" was exclusively granite brought across Massachusetts Bay, first from quarries in Maine and afterward from near Cape Ann. It was shipped in schooners, and on arrival at the canal was transferred to lighters and thence placed by floating derricks. Weight was computed by vessel displacement, as provided in the specifications, payment being made per ton of 2 000 lb. in place in the breakwater.

The only damage that the breakwater has sustained was during the high gale on January 13th, 1915, when stones above mean sea level at the outer end, for a distance of about 200 ft., were removed by the waves. The damage began at the end and then continued by successive cutting back, stones being knocked off separately as each in turn found no support on one side. The damage was repaired by using larger boulders and by building on somewhat flatter slopes at the end, the previously displaced boulders lying on the flanks giving a broad base on which to build.

Rip-rap.—The banks of the canal, consisting of sand and gravel, required protection against wave action, which protection it was decided should be provided by loosely deposited material rather than any form of paving. Accordingly, the specifications were drawn to read:

> After any section of the canal has been constructed to its finished prism long enough for all slips, slides, and settlements to have taken place, and as soon as the banks have assumed a permanent and stable position the Engineer may order the construction of the wash walls or revetment. This wall will be constructed within the limits shown in Drawing No. 6 and will consist of a rough pavement, dumped into place and 18 in. thick, of one-man stone.
>
> Small inequalities in the alignment or slope of the bank shall be leveled up and made smooth before the placing of the wash walls.

In actual practice the stones varied in weight from 5 to 150 lb. each.

Fig. 3 shows how the rip-rap was placed with respect to the water surface as shown on Contract Drawing No. 6, referred to in the specifications. In general, it extended from a point about 6 ft. above mean high water to 6 ft. below mean low water, which gave a protected sur-

face 42 ft. wide, measured on the slope at the east end of the canal, where the tidal range is greatest.

At the western end of the canal, from Station 360 to the north of the Monument River, rip-rap has been omitted, as not being deemed necessary because the estuary of the river, here canalized, gives a wider water surface, and the banks are protected naturally by tough marsh grass. Experience has confirmed this assumption.

The rip-rap is of granite. That used in the easterly half came from the quarries that furnished the large stones for the breakwater; some of the stone at the westerly end came from the quarries on Long Island Sound, but, after the last dam was cut and traffic was opened through the canal, the remainder of the stone came from Cape Ann.

About one-half of the cut made by the Foley Company was rip-rapped in the dry by that company with broken boulders from the excavation, the remainder being done by the general rip-rap contractor after water was admitted.

The stone which was brought to the work came in schooners and was transferred to lighters. It was dumped on the banks from scale boxes holding about 2 tons each, and was spread by hand at low water, though the derrick men soon became sufficiently expert to do considerable of the spreading by sweeping the scale boxes up the back.

The quantity of rip-rap placed was 144 397 tons.

It was expected that, when the operation of the canal began, the effect of waves from passing vessels might scour the banks below the rip-rap so as to cause the latter to slide down, and require additional material to be added to the top. This possibility of easy repair is one of the great merits of loosely placed rip-rap, which quality also tends to localize damage if it settles at any point, the loose stones quickly finding a new bed. Rigid pavement, on the other hand, may resist falling until much sand has been displaced behind it, and then collapse over a large area. Other advantages are low first cost, rapidity of execution, and rough surface to check wave action and tidal current.

After 30 months of operation only very trifling repairs have been made. This permanent condition of the rip-rap is due partly to the naturally hard condition of the material composing the banks and partly to the depth to which the rip-rap has been carried, apparently below serious wave action.

Work on the breakwater was begun in June, 1909, and was finished in December, 1913, in 35 months of actual working time. It could have been completed earlier had there been necessity. Progress in placing rip-rap was naturally governed by canal excavation, but was at the maximum in 1914, when 8 400 tons were placed in 1 month.

Railway Reconstruction.—The Old Colony Railroad Company, now leased to the New York, New Haven and Hartford Railroad Company, owned the line from Boston to the Cape. It was a double-track line from Boston to the station known as Buzzards Bay, where it forked into two single-track lines, one crossing the Monument River at Buzzards Bay and running to Woods Hole, the other turning east and running up the valley of the river and thence along the hook of the Cape to Provincetown. The latter line not only occupied a portion of the lands required for the canal location, but actually crossed the canal line three times.

The Canal Company's charter contemplated the taking of the Railroad Company's right of way, under proper safeguards to the latter's operation. Conferences with the engineers of the New Haven and the Old Colony Railroad Companies—for the latter still maintains its full corporate existence—resulted in an agreement:

- To relocate both branches from Buzzards Bay station, and, though leaving the actual physical junction at the station, to make a single crossing of the canal for both lines, with the point of divergence on the south bank;
- To relocate a short piece of the Woods Hole line from the point of divergence to connect with the existing line;
- To relocate the Provincetown line along and near the south side of the canal right of way until it intersected the old line east of Bourne, and again to relocate a piece at Bournedale, where the railroad, in order to avoid heavy cutting, crossed and re-crossed the canal location.

Previous to these changes, the junction of the lines was at the north end of the Buzzards Bay station, but after the relocation was completed the junction point was at the south end of the station, and this change required a complete reconstruction of the Buzzards Bay yard. The old junction and yard switches, with their signals, were controlled by lever operation, a system that could not be altered to in-

clude a draw-bridge with its home and distant signals on the far side of a waterway all interlocked and controlled from a central tower. The mechanical system, therefore, was abandoned and a new electro-pneumatic plant put in. All told, there were 6.3 miles of single main track and 1.2 miles of side track newly constructed. The quantity of track work was not large, but the expense was proportionately high, as the yard, signals, water supply, and bridges brought the total cost to \$379 274.05, in addition to which the Railroad Company bore the expense of a new station building and certain extensions of the signal system that were not thought to be called for by the presence of the canal. The only items in the railroad construction that are of interest are the main drawbridge and other bridges, and these are described in detail. The contract for railroad work was taken by the Degnon Contracting Company and sublet to the Wilson and English Construction Company.

Highways.—The highways proved an annoying detail to adjust. Those existing before the canal construction began were wholly incompatible with the operation of a ship waterway, there being no less than seven crossings of the location, and if any of these crossings were abandoned, certain pieces of property would be left without access. After many local public hearings, and discussions with the County Commissioners and Selectmen of the towns, it was finally decided to construct highways on both sides of the canal, so as to provide continuous east and west facilities, and to restrict the actual crossings to three, at the Villages of Bourne, Bournedale, and Sagamore. The decision as to whether these crossings were to be by bridge, ferry, or tunnel was by the charter left to the Joint Board, who ordered bridges at Bourne and Sagamore, and a ferry for passengers only at Bournedale. These bridges will be considered in connection with the railroad bridge.

New highways, 4.4 miles in length, including the portions occupied by bridges, were constructed, and 0.6 mile was resurfaced, all of which the local authorities insisted should be built to a standard never before observed in that part of Massachusetts, involving heavy earth cuttings and embankments.

Fig. 5 shows the railroad and highways as they existed before and after relocation, with the points of crossing.

Bridges.—There are three bridges across the canal. The charter requirements required the canal to have a prism with a bottom width of 100 ft., but the directors realized that it would be only a question of a short time when a wider canal would be required—one sufficiently broad to permit tows to pass in opposite directions, and a deeper canal to accommodate battleships or vessels drawing more than 25 ft. Although the canal can be deepened or widened at any time, bridges are rigid obstructions that can be altered only by complete reconstruction. The directors, therefore, decided to build in the first instance bridges larger than were demanded to meet the requirements of the charter, and large enough to fit a canal of such increased dimensions as could be reasonably foreseen as probably necessary. After considering various designs and estimates, channel draw-spans of 160 ft. in length, from center to center of piers, were adopted, with foundations to a depth permitting the canal to be made more than 30 ft. deep.

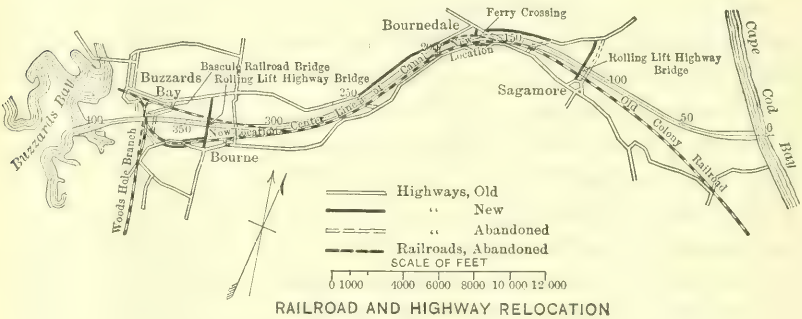
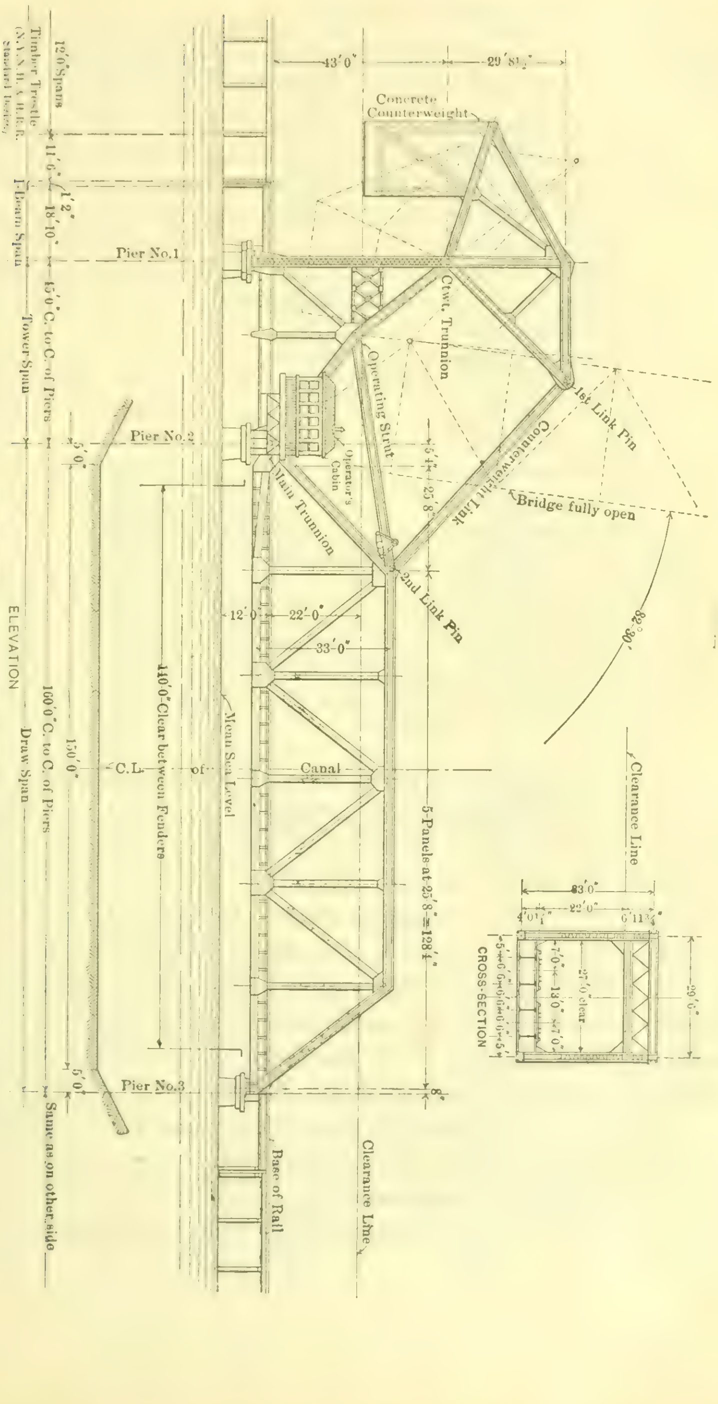


FIG. 5.

Swing bridges were out of the question, as central piers would completely block a straight channel. For the railroad bridge, after conferences with the engineers of the New Haven Railroad Company, the Strauss type of trunnion bascule bridge was selected, to be built to meet the New York, New Haven and Hartford Railroad bridge specifications of 1908, which contemplate two engines of consolidation type with 60 000 lb. per driving axle, 30 000 lb. leading axle, and 39 000 lb. per tender axle, or 65 000 lb. per axle on two axles 7 ft. apart for details. The general elevations and cross-section are shown on Fig. 6.



ELEVATION

BUZZARDS BAY RAILROAD BRIDGE

FIG. 6.

Special specifications to cover the design of a bascule bridge of this type, and not contemplated by the railroad specifications, were added, of the important clauses of which the following is a summary:

Impact of motion: $33\frac{1}{3}\%$ of dead-load stress in structural steel members during the motion of bridge to be added.

Operating machinery: This designed to withstand a wind pressure of 20 lb. per sq. ft. of the moving leaf, and the motors strong enough to open or close the bridge through the full angle of the opening in $1\frac{1}{2}$ min. against a wind pressure of 5 lb. per sq. ft., or in 2 min. against a wind pressure of 15 lb. per sq. ft.

Machinery unit stresses:

Trunnions and pins	15 000 lb. per sq. in.
Slow-speed shafting	16 000 " " "
High-speed shafting	12 500 " " "
Slow-speed gears	17 500 " " "
Medium-speed gears	9 000 " " "
High-speed gears	9 000 " " "

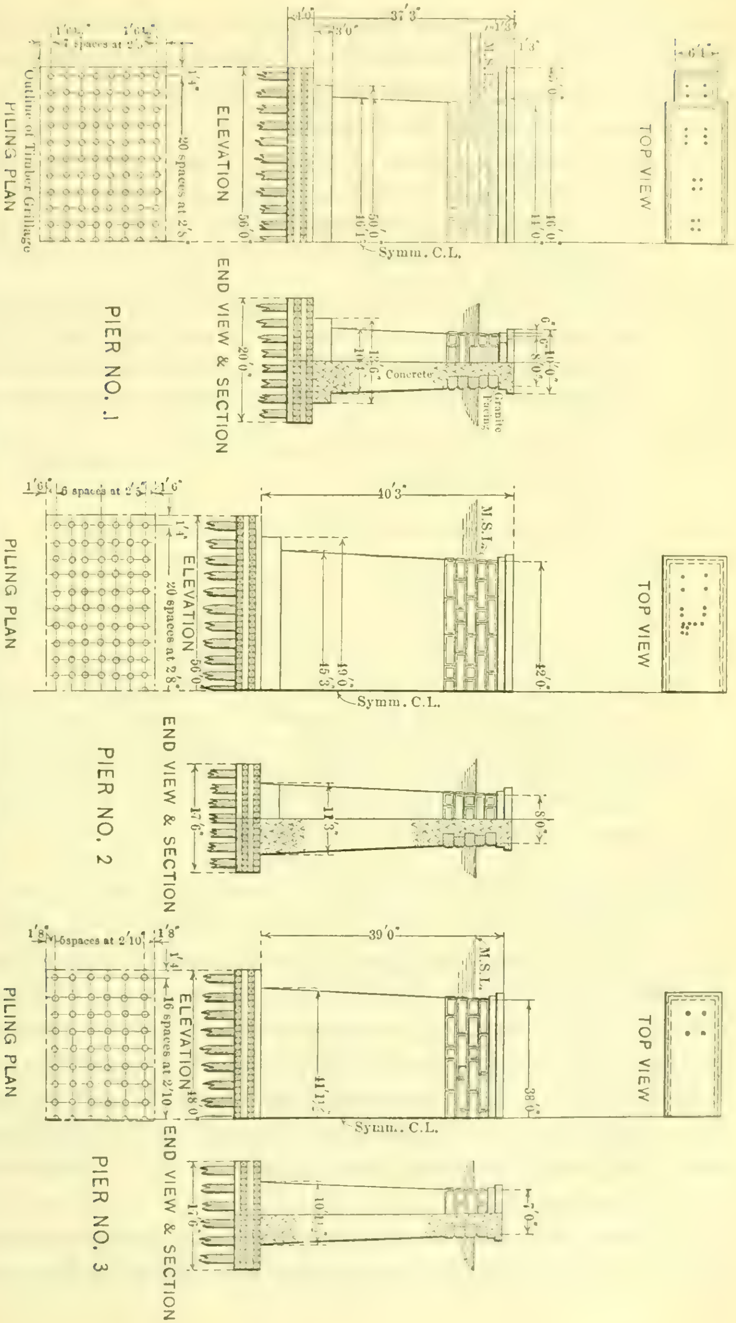
Bearing in journals during motion:

Trunnions	1 700 " " "
Slow-speed shafting	500 " " "
High-speed shafting	500 " " "

Motors: The main operating motors are two 65-h.p. direct-current motors, capable of carrying an overload of 50% for 10 min. and 100% for 5 min.

For the substructure of the Buzzards Bay Bridge the Chief Engineer of the New Haven Railroad Company requested that concrete should not be exposed to the action of salt water, especially between high and low water, on account of the generally unsatisfactory experience with this material in Boston Harbor and vicinity, and that the piers should be lined on the outside with durable stone.

Fig. 7 shows the main details of the three piers. There were no masonry abutments, as the connection from the draw-span to the shore was made by wooden pile trestles of the New Haven Railroad standard. The piers were numbered in order from the north end: 1, 2, and 3, the canal flowing between Piers 2 and 3. The maximum load falls on Pier 1, due to the counterweight. Piers 2 and 3 divide the live load, but the former supports a greater portion of the weight of the truss. The weights carried by the three piers, therefore, vary greatly.



PIERS FOR BUZZARDS BAY RAILROAD BRIDGE

Fig. 7.

On account of the large area needed for the bases of the piers, especially of Pier 1, it was decided to excavate by dredges to the necessary depth and place an open caisson on piles.

The specifications required, in general:

That the facing should be of granite in 18 to 20-in. courses, to the under side of the coping, which was to be of concrete.

The piles were to carry not exceeding 24 tons each, with a minimum diameter of 8 in. at the tip and 12 in. at one-third the distance from the butt.

The concrete in the foundations or hearting was to be mixed in the proportion of 1 part cement, 2½ parts sand, and 5 parts broken stone or gravel, in which boulders might be embedded. In the concrete in the coping course the proportions were 1: 2: 4.

The cement was to be of an accepted brand, and to give the following tensile strengths:

Tensile Strength, in Pounds per Square Inch.

Mixture.	24 hours.	7 days.	28 days.
1:0	175	500	575
1:2	—	225	300
1:3	—	175	225

A minimum increase of 12% from 7 to 28 days was required for neat briquettes, and 20% for sand briquettes mixed 1: 2.

The maximum content of anhydrous sulphuric acid (SO_3) was 1.75%, and of magnesia (MgO) 3%, and no addition greater than 3% to the ingredients making up the cement subsequent to calcination was permitted.

The contract was taken by the Holbrook, Cabot and Rollins Corporation, of Boston, and work was begun on November 9th, 1909.

A clam-shell dredge was brought through the old railroad trestle, and dug the three pits, so that the piles could be driven. Although a jet and hammer were used in combination, no greater penetration than an average of 19 ft. could be made with the piles for Pier 1, and 15 ft. for Piers 2 and 3, on account of the very dense sand and gravel. The specifications called for 20 ft., and, to make up for the deficiency, twenty-two extra piles were driven under Pier 2. The details are clearly shown on Fig. 7.

The elementary features of the superstructure, which is a Strauss bascule, are fixed trunnions and parallel link motion of the counter-

weight. Equilibrium is maintained by four pins forming the corners of a parallelogram arranged so as to make the angular movement of the counterweight frame equal to the angular movement of the movable span. The reactions of the main trunnion pier and the counterweight trunnion pier are both always vertical, as the horizontal components neutralize each other in any position of the bridge.

The structural parts of the bridge consist of: The movable single-arm channel span, *A* (Fig. 8), the counterweight frame, *B*, rigidly connected to the concrete counterweight, *C*, the connecting link, *D*, the operating strut, *E*, and the tower, *F*, supporting the counterweight.

The mechanical parts consist of the operating machinery, the main trunnions, the counterweight trunnions, the link pins, and the auxiliary apparatus.

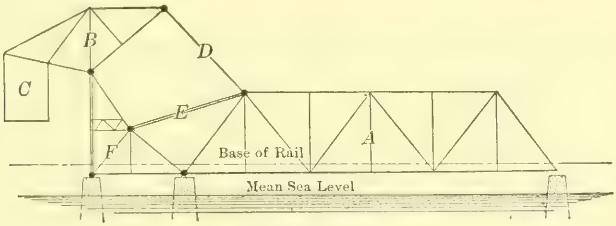


Fig. 8.

The movable channel span consists of two Warren trusses rigidly framed together by the floor system carrying a double-track railroad. The length of span is 160 ft. from center to center of piers, the trusses being 29.6 ft. from center to center. The elevation of the base of rail is 12 ft. above mean sea level, and the clearance line for the trains is 22 ft. above the base of rail.

The counterweight frame is proportioned so as to keep the movable span and the counterweight in equilibrium in any position of the bridge. The frame consists of two trusses, between which the concrete counterweight is placed. The resultant load of the counterweight and the link stress is taken by two trunnions, each supported by two bearings at the apex of the tower. Special collars riveted to the frame are bored with a driving fit, and the trusses are fastened to the trunnion with tapered keys. The counterweight is attached to the rear arm of the truss, and rigidity is attained by extending the structural framing into the concrete. Besides the weight of

the concrete, pockets are provided for shaped iron castings, which allow for close adjustment in balancing the bridge. The link strut consists of two posts, pin-connected at the top to the front arm of the counterweight frame, and at the bottom to the hip of the movable span. Lateral stiffness is secured by diagonal and horizontal bracing. The link, at any angle of the movable leaf, is always parallel to a line which travels through the center of the main and counterweight trunnion.

The operating struts to which the main rack is attached are pin-connected at one end to the tower, and at the other are engaged to the main operating pinion. A special yoke, mounted on each side of the main pinion, serves as a guide for the strut during operation.

The tower supporting the counterweight consists of two main columns, a diagonal strut between the two trunnions and a horizontal strut at the base. Inclined posts are supported at the base on a transverse steel truss embedded in the masonry, giving, in addition to portal bracing at the top, great lateral stiffness.

The wind pressure against the floor system of the raised bridge is carried to the main trunnions. By the horizontal strut at the base of the tower, together with the floor system, the reaction is divided between the two piers. The safety against overturning is thus increased, and the corresponding eccentric loading on the foundations is materially reduced. All trunnions and link pins are of forged steel. The bearings and caps are of cast steel and lined with phosphor-bronze bushings. Helical oil grooves are cut in the caps and bearings for the proper distribution of the lubricants.

The motive power for operating the bridge is electricity. Two 65-h.p. electric motors operate under a 550-volt direct current. The power is transmitted to the main operating pinion by a train of gears. In addition to the electric brake on the motor, there is also a motor-driven emergency brake. Special equalizing gears regulate the uniformity of speed between the duplicate system of operating machinery. The motors and gearing are on the movable span, and are protected by special housing. A separate motor with the necessary mechanical connections operates the locking device.

All machinery and auxiliary apparatus are operated by controllers, switches, switchboards, indicators, and instruments conveniently placed in the operator's house. From this point the operator has per-

fect control of all traffic passing through the canal and over the bridge. The control of the bridge is interlocked with the main tower controlling all signals and switches in the Buzzards Bay yard, although arranged so that the tower operator can free the bridge for independent operation.

The highway bridges at Bourne and Sagamore are of the double bascule type, of the same dimensions and almost identical design. It will be convenient, therefore, to treat the substructures and superstructures of these bridges together.

The substructure for the highway bridge at Bourne was designed on quite different lines from that of the railroad bridge at Buzzards Bay, a design that was followed with few changes in details for the highway bridge at Sagamore. On account of the great lightness of the highway bridge, there was no need for such a massive substructure as was built at Buzzards Bay, and any attempt to give an exterior protective surface of cut stone would have been very expensive. The writer is one of those who believe that damage to concrete by the action of salt water is both chemical and mechanical, and that the destruction can be greatly retarded, if not actually prevented, if the concrete is allowed not only to set, but to become quiescent chemically, before exposure to the rise and fall of the tides.

The most economical type of substructure was an isolated support beneath each point of load, varying in diameter according to the load intensity. Such a type was selected, and in order to keep the surface of the exposed piers from tidal exposure until internal changes had ceased, the specifications required that the forms for the concrete should be of surface lumber and left in place. As the teredo is very active, creosoted lumber was called for. The intention was to let the protective wood remain until it was destroyed by natural agencies, or for at least a year, and then to take it down. As a matter of fact, it is still in place and in good condition, both at Bourne and Sagamore. The lumber being matched and not unsightly, it has been allowed to remain. The concrete is now more than 5 years old, and, as shown by recent borings through the lumber, is in perfect condition. The writer believes that, if the lumber should be removed, no injurious effect would now result from the action of salt water during either the summer or winter.

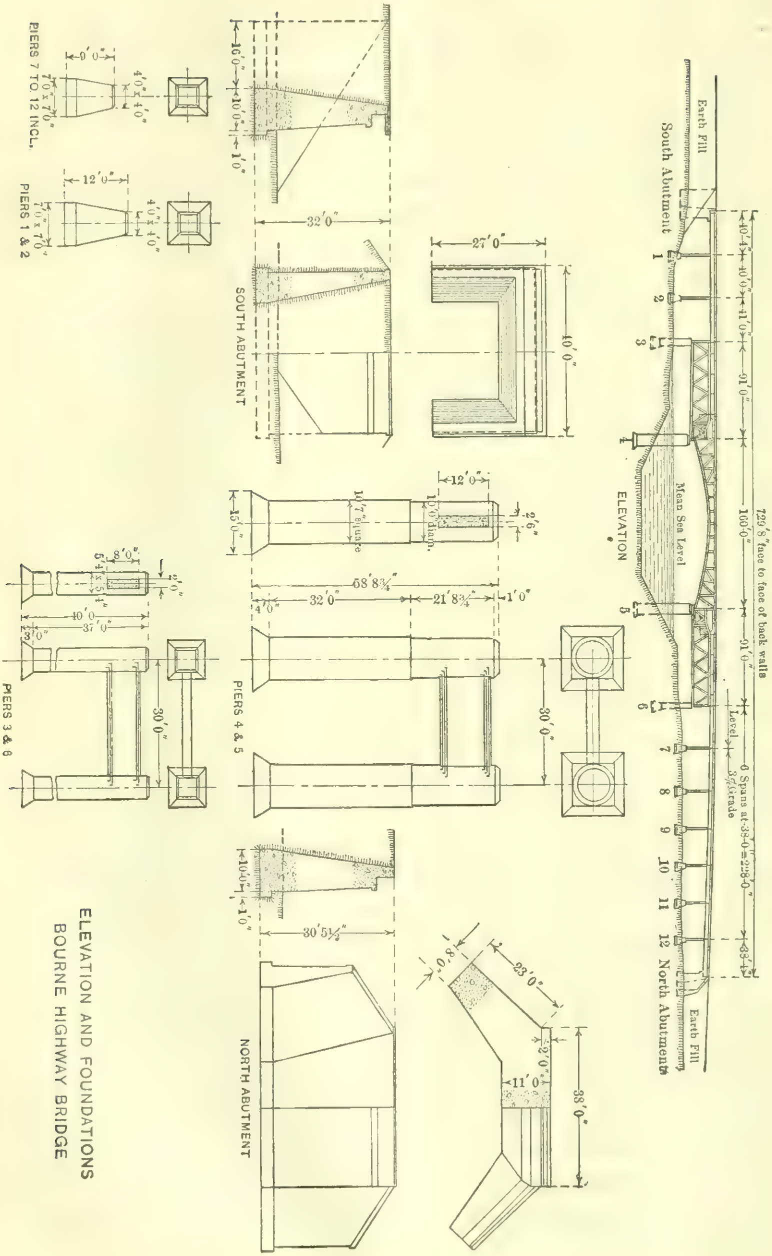


FIG. 9.

ELEVATION AND FOUNDATIONS
BOURNE HIGHWAY BRIDGE

The specifications were similar to those for the foundations of the Buzzards Bay Bridge, except that no piles were contemplated, and pressure on the sand was limited to 3 tons per sq. ft. The requirements for cement and concrete were the same, a mixture of the latter in the proportion of 1 : 2 : 4 being used in the upper 2 ft. of the main and secondary piers.

Fig. 9 shows the general elevation of the Bourne highway bridge and details of the substructure. The main piers, Nos. 4 and 5, and the secondary piers at the ends of the approach girders, are square in section, 15 ft. 1 in. and 8 ft. 10½ in. on bases, respectively, and 10 ft. 7 in. and 5 ft. 4 in. on the shafts to Elevation 96, and above that elevation, circular. In order to stiffen these piers laterally, reinforced concrete girders were designed for all piers, these girders being 12 ft. deep and 2 ft. 6 in. wide in the case of the main piers, and 8 ft. deep and 2 ft. wide for the secondary piers.

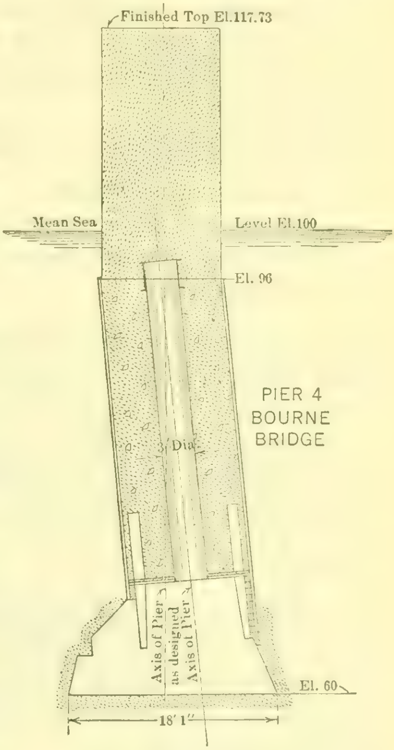


FIG. 10.

All the caissons for these eight piers were sunk by compressed air and then filled with concrete, the material through which they passed being for the most part fine sand. Pier 4 (the westerly one), just before reaching grade, was badly deflected by striking a boulder. One of the cutting sides of the caisson was cut away, and, under cover of poling boards, the sand was excavated unsymmetrically, as shown in Fig. 10, so as to bring the axis of pressure well within the middle third of the base, the increased area of base reducing the pressure at the outer edge to proper limits. Above Elevation 96 the pier was carried straight up. The heavy reinforced girder connecting the tops of the two main

piers affords sufficient rigidity to overcome the objection to the inclination.

The work was executed by the Holbrook, Cabot and Rollins Corporation, the layout of the erecting plant being shown by Fig. 11. Access to both sides of the river, the canal not being dredged at the time, was had by a temporary trestle, carrying a traveler and also a concrete car. A second traveler on the ground covered the piers at the north end.

For the Sagamore Bridge the contract for the substructure was taken by the Degnon Cape Cod Canal Construction Company and sublet to the Dravo Contracting Company of Pittsburgh, under date of April 1st, 1912.

The general plan called for 2 abutments and 26 piers:

Two pairs of main piers,
Two pairs of piers supporting approach girders,
Nine pairs of small piers on land,
Two abutments.

The specifications were substantially the same as those for the Bourne Bridge, except that piles beneath some of the piers were contemplated.

The foundations, where on sand or gravel, were to carry a load of not more than 3 tons per sq. ft., and, where on piles, a load not exceeding 20 tons per mile, as determined by the *Engineering News* formula, $\frac{2\,Wh}{s+1}$.

The forms on the main piers were to be left in place, as with the Bourne Bridge, but the creosoting of the lumber was omitted, long-leaf pine, dressed to $2\frac{7}{8}$ in., being called for. At the end of 3 years the forms are still in place, and the concrete, as at Bourne, is in perfect condition.

For the caissons for the pair of main piers, No. 10, the contractor designed a steel shell of $\frac{1}{4}$-in. plates, with the lower part 16 ft. 6 in. in diameter for a height of 10 ft., then reducing through a height of 4 ft. to a diameter of 12 ft. $5\frac{1}{2}$ in. for a height of 36 ft., and there the wooden form began with an internal diameter of 10 ft. The shell weighed about 16 tons, and was sunk by removing the material from the inside with an orange-peel bucket. At Elevation 56.5 the

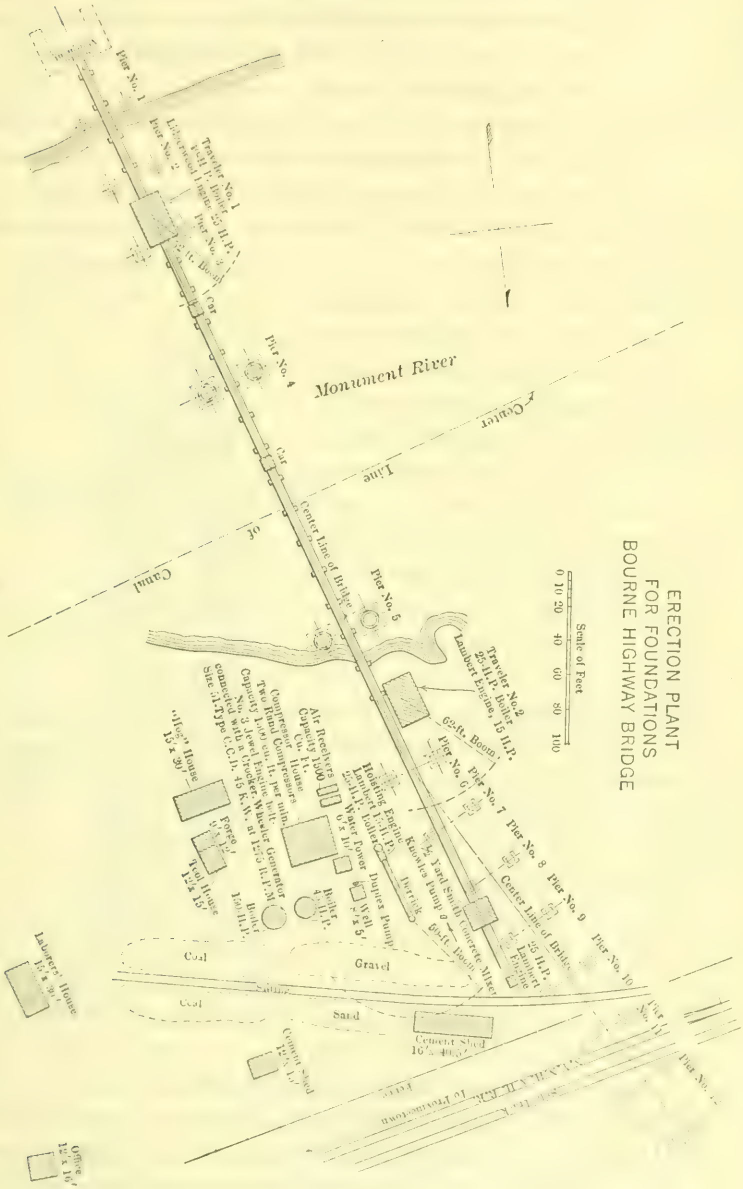

ERECTION PLANT
FOR FOUNDATIONS
BOURNE HIGHWAY BRIDGE
Scale of Feet
0 10 20 40 60 80 100
Monument River
Center Line of Canal
Center Line of Bridge
Pier No. 1
Pier No. 4
Pier No. 5
Pier No. 7
Pier No. 8
Pier No. 9
Traveler No. 2
25-H.P. Boiler
Lambert Engine, 15 H.P.
62-ft. Boom
Air Receivers
Capacity 1500
Cu. Ft.
Compressor House
Coal
Gravel
Sand
Siding
Cement Shed
Laborers' House
Office
12' x 16'

FIG. 11.

bottom was sealed by divers with concrete in bags to a height of 5 ft. Then 12 ft. of concrete were deposited under water in buckets, and after this had set the water in the shell was pumped out and the remaining concrete was placed in the dry.

Main piers No. 9 were sunk by the pneumatic process. The steel shell at the bottom was splayed at the foot to a diameter of 12 ft. 6 in., and lined with concrete, as shown in Fig. 12. The cutting edge was carried down to a gravel bed at Elevation 55, but concrete was carried down 0.4 ft. lower.

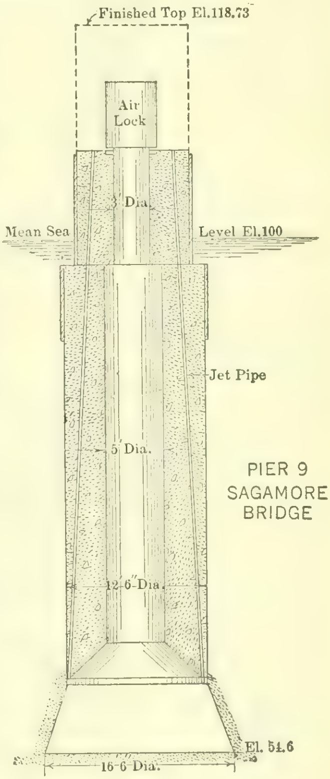


Fig. 12.

For Piers Nos. 8 and 11, supporting the ends of the 90-ft. approach girders, steel shells, 6 ft. 6 in. in diameter, were sunk by removing the material within the shells with an orange-peel bucket. In Piers No. 8 six 25-ft. piles were driven, reaching below Elevation 72, with concrete beginning at Elevation 92. The similar piers on the other side of the canal were put down in the same manner, except that concreting began at Elevation 91.

The abutments were founded directly on hard sand, but the concrete in the small pedestals, 1, 2, 3, and 4, rested on piles cut off below ground-water level, as requested by the owners of the car manufacturing plant on whose property they were placed, as they were fearful of damage arising from the known presence of quicksand.

The main piers, Nos. 9 and 10, and secondary piers, Nos. 8 and 11, are connected by reinforced concrete girders, as at Bourne.

The Sagamore Highway Bridge crosses the waterway about 2 miles west of the easterly entrance to the canal, and, as a structure, is divided into three sections: the main channel span, the north, and the south approach.

The elevation of the roadway above mean sea level is 41 ft., so as to give clearance for medium-sized vessels without opening the bridge, while also providing clearance for the New York, New Haven and Hartford Railroad passing under it at the south end of the steel structure approach. Part of the south approach on earth fill and of the north approach in steel has a 5% gradient.

The main-channel span is divided into three parts: the movable section, and the two stationary approach trusses. The width of the bridge between railings is 30 ft., divided into a 25-ft. roadway and a 5-ft. sidewalk. A single-track electric railway is also carried across, with the necessary overhead construction.

The movable section consists of a Scherzer, double-leaf, rolling-lift bridge, of 160 ft. span. The characteristic features of this bridge are two 80-ft. cantilever spans, the rear end of each truss being extended into a segmental girder which rolls on a track girder of special heavy construction.

To keep the movable part in balance, a concrete counterweight is rigidly attached to and framed in between the two lines of trusses, forming the main supporting members of the rolling-lift span. The track girders are incorporated in the framework of the approach trusses, one end resting directly on the main pier, the other being framed into a carrying girder placed at the near end panel point of the approach trusses. The entire dead load of the movable span is transferred to the contact surface between the curved segmental girder and the cast-steel plate of the track girder. Special lugs on the track girders, which fit into corresponding recesses in the curved section, insure perfect travel of the bridge in the vertical plane. As the bridge opens it rolls backward on the track girder, leaving a clear channel for passing vessels.

The trusses of the movable leaf are designed with a straight top chord; the bottom chord is slightly curved from the point of support toward the end of the cantilever. The three end panels are built up with a solid web; the three panels next to the support are of open-web construction with vertical and diagonal built-up sections. The floor system and the top laterals, together with horizontal struts placed between the trusses, provide against lateral distortion of the bridge in any position.

The approach trusses, each of 90-ft. span, are of the Warren type with stiff riveted web members. The trusses are spaced so as to provide proper clearance for the receding counterweight as the bridge opens. Any danger of damage resulting from the upward reacting force at the rear end of the lift span, due to unbalanced live load or impact in closing, is guarded against by a bumping girder placed on the top chord of the approach span. The ends of each movable leaf are provided with a sheer lock designed with male and female parts, which in a closed position are interlocked, so as to insure proper alignment and additional lateral stiffness.

The north and south approach spans are in lengths of 38, 50, and 75 ft. The approach spans are of conventional design, with the two main girders seated on top of the columns, laterally connected by horizontal struts and diagonal bracing. The floor-beams throughout the entire length of the bridge are about 13 ft. apart, and are framed into the web of the main trusses or girders. On the channel approach trusses, they rest on the top chord. The roadway stringers consist of 9-in. I-beams about 2 ft. 6 in. from center to center, except for the support of the railway track, where 10-in. I-beams are placed in pairs under each rail. Bolted to the top flange of each stringer are nailing strips, to which is fastened the 4-in. oak plank decking, being laid with ¼-in. joints and sufficiently crowned to provide for quick run-off of rain water. The sidewalk brackets are in line with the floor-beams, and carry wooden stringers to which the 2-in. plank flooring is fastened. On the lift span there are special fastenings to guard against possible movement of the floor system when the bridge is being raised. The hand-railing is of 2-in. gas pipe posts, from 6 to 7 ft. apart, and joined by two lines of 1½-in. pipes at the top and near the base of the posts, the panel between the two lines of pipe consisting of a grillwork of light angles and bars.

The motive power for operating the bridge is electricity, furnished at 550 volts, each movable leaf being operated by a 25-h.p. direct-current motor. The power transmission from the motor to the main operating pinion is arranged by gear trains and countershafts mounted on the movable span. The horizontal rack which engages the main pinion is attached to the approach trusses with the pitch line parallel to the horizontal top flange of the track girder.

There are safety gates on the approach trusses directly in front of the break in the floor system; they are opened and closed from the operator's house. From this point the bridge tender has under his control all the electrical apparatus for the operation of the machinery, signals, lights, and auxiliary mechanical devices for the protection of the structure itself, as well as the regulation of traffic over the bridge and through the canal.

The Bourne Highway Bridge, which crosses the canal about 1½ miles east of the westerly entrance, is designed along the same lines as the Sagamore Bridge, the only difference between the two structures being the length of the approaches and the width of the roadway, the latter, for the Bourne Bridge, being 30 ft. The preceding description of the Sagamore Bridge covers in a general way the structural features of both bridges.

Fenders.—All the bridges are protected with fender work. At the easterly end of the canal there are no destructive marine borers, and the fender for the Sagamore Bridge at this end of the canal was built of untreated wood. In the warm water of Buzzards Bay the marine borers are very destructive, and the fenders for the two bridges at this end of the canal are of creosoted pine. The approach portion of the fenders was designed and built in the ordinary manner with clusters of piles at the extreme ends and three rows of staggered piles in the approach proper. In the narrow throat immediately adjacent to the piers the common design was abandoned in order to obtain the maximum width between the fenders. This portion of the fender was built with 50-ft. span vertical trusses, in the case of both highway bridges, and with 60-ft. trusses for the railway bridge. These trusses were designed of sufficient strength to carry their own vertical load and with sufficient lateral stiffness to stand the rubbing impact of vessels. This lateral stiffness is provided by four courses of 6 by 12-in. timber wales, blocked, braced, and spliced, as shown by Fig. 13. The first and third courses from the channel side are extended and fastened to three of the dolphin piles to which collision impact is ultimately transmitted.

Fig. 13 shows the general design of the Buzzards Bay railway bridge fender, the design for the other bridges being similar.

Lighting.—The canal proper is lighted by 7-volt, 6.6-ampere, or 32-c-p., incandescent lights. These are carried by iron brackets on wooden poles, the poles being set at about the level of mean tide, with the plane of the lights 25 ft. above mean low water. The poles on the south side of the canal are painted red; those on the north side are painted black, thus following the rule for the colors of buoys entering a harbor, the canal being considered as an entrance to Boston Harbor. These lights are operated on an alternating current of about 500 volts, and are connected in series.

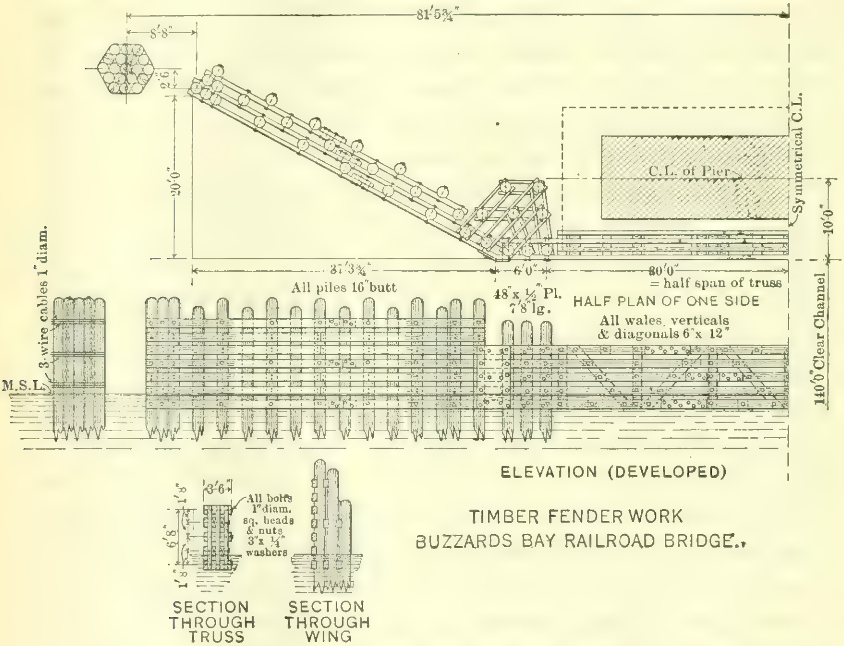


FIG. 13.

In the Buzzards Bay approach channel there are seventeen lights, maintained by the Lighthouse Department of the United States Government, the approach being in the open navigable waters of the United States. Four of these are on buoys at the western end of the channel and thirteen are on pile dolphins marking the channel from Wings Neck to Agawam Point. Coming from the west the lights on the port hand are white and on the starboard hand red. All these lights burn acetylene gas, flashing about 3 sec. duration and 2 sec. eclipse. On one of the two buoys at the western entrance there is a bell which

rings by wave action. In time of fog there are also three bells operated by storage batteries, the bells being maintained by the Canal Company.

On the south breakwater there is a 250-watt 111-volt incandescent light with a stereopticon globe, giving about 16 000 c-p. The plane of this light is 40 ft. above the water surface. At the eastern entrance there are two lights on buoys outside the breakwater, or about 400 ft. from its end, with a red flash, being luminous for 5 sec. and a gas and bell buoy 2½ miles northeast of the breakwater, showing a white flash every 6 sec., with duration of 2 sec., the light being 390 c-p.

Electrical Power.—The electrical power for the lighting of the canal and for the operation of the three bridges crossing it is purchased from the Southeastern Massachusetts Power and Electric Company, and is transmitted from steam plants at New Bedford or Plymouth. The power is transmitted at 22 000 volts, and can be supplied from the two sources by either of four routes to a step-down transformer sub-station near the west approach of the Bourne Highway Bridge. In the sub-station there are two 20-kw. alternating-current, gasoline-engine driven generating sets, which can be started on short notice for emergency conditions. From the sub-station 2 300-volt lines connect the three bridges, and, in addition, standard street lighting equipment is provided for the lights on each side of the canal.

At the western end of the Buzzards Bay Bridge there is a storage battery of 280 cells, of the chloride accumulator type, which is capable, at full charge, of supplying sufficient power to open and close the Buzzards Bay Bridge twenty-five or thirty times without replenishment.

In the operators' cages on each of the bridges there is a motor generator set for generating 550-volt direct current for operating the moving parts of the bridges and for charging the battery. In case of failure of the alternating current supplied over any one of the four transmission routes, power is available from the two gasoline-engine generating sets, or from the storage battery.

In addition to the foregoing sources of power, the street railway feeders of the New Bedford and Onset Street Railway Company can supply power at the Bourne and Buzzards Bay Bridges. Also, a special connection has been made to the Keith Car Company's plant at Sagamore, for the operation of this bridge in an emergency.

The present policy of operation in an emergency is to rely on the storage battery. There are separate connections from the battery house to the operator's cage on the Buzzards Bay Bridge, and, through special switching arrangements, the alternating-current lines connecting the three bridges are used in parallel for the positive side of the battery for the operation of the bridge motors, and the canal channel is used for the negative return. The capacity of the battery is ample to operate all three bridges many times without replenishment.

The power for operating the bridges and lighting the canal runs from 6 000 to 8 000 kw-hr. per month at a maximum peak load of about 140 kw.

The Buzzards Bay Bridge requires about 100 h.p. of maximum demand and about 1 min. to open or close. The power required for one operation is about 2 ampere-hours. The Bourne and Sagamore Bridges each require about 20 h.p. for the maximum demand and about the same time to open or close as the Buzzards Bay Bridge.

Engineering Personnel.—The responsible engineers of the company who have had charge of the work have been, of the Canal Company, W. J. Douglas, M. Am. Soc. C. E., Deputy Chief Engineer since August, 1915, and of the Construction Company, Eugene Klapp, M. Am. Soc. C. E., from the commencement of operations. The engineers in charge on the ground have been Henry W. Durham, M. Am. Soc. C. E., Resident Engineer from the commencement of operations to April 1st, 1912. Charles T. Waring, Assoc. M. Am. Soc. C. E., Resident Engineer from April 1st, 1912, to August 1st, 1914, when the canal was declared officially opened, although construction was not quite complete, and when Mr. Waring became Superintendent. From August 1st, 1914, to January 31st, 1915, the completion of construction was in charge of A. S. Ackerman, Assoc. M. Am. Soc. C. E., previously Assistant Engineer, when the duties were re-assumed by Mr. Waring in connection with his other duties as Superintendent, and which he retained until January 30th, 1916. Mr. W. S. Crocker, who was Assistant Engineer during construction, has become Resident Engineer under the operating management.

To these gentlemen and to their assistants the Chief Engineer expresses his obligations, and, in the preparation of this paper, particularly to Mr. Eugene E. Halmos, member of the staff, for his co-

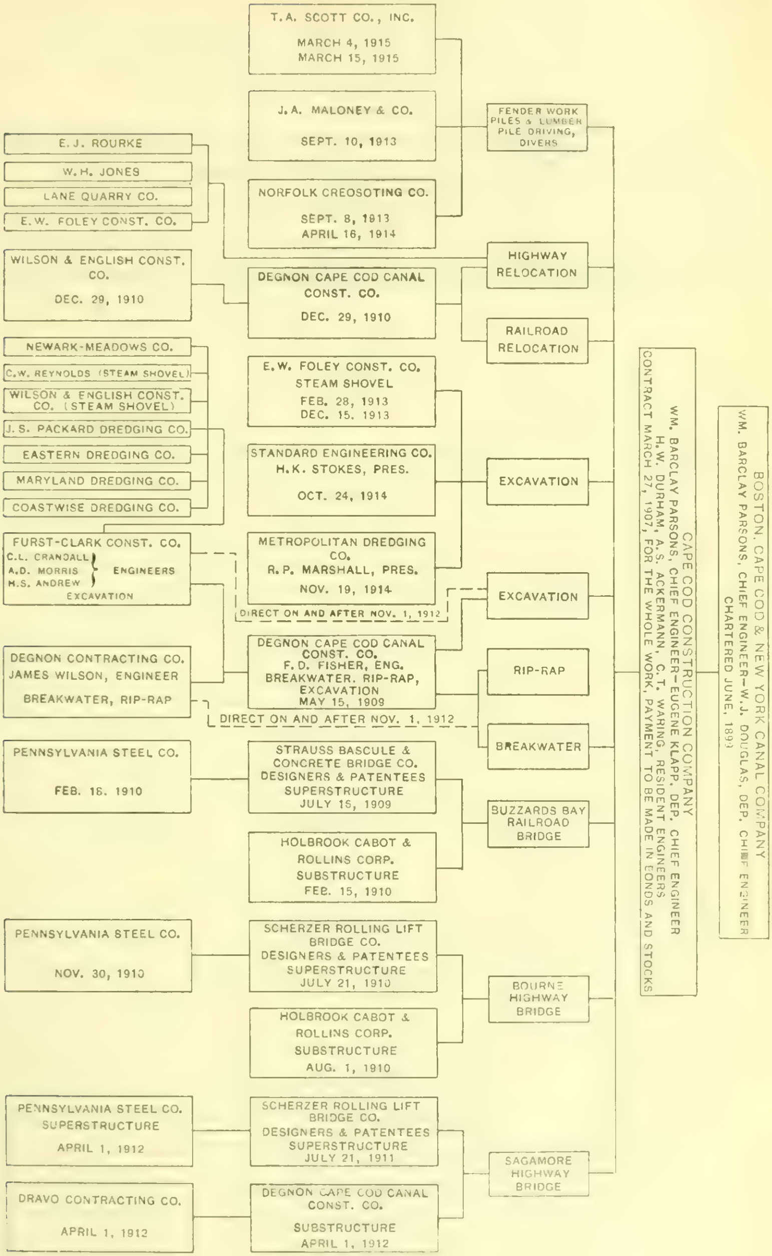

T. A. SCOTT CO., INC.
MARCH 4, 1915
MARCH 15, 1915
J. A. MALONEY & CO.
SEPT. 10, 1913
NORFOLK CREOSOTING CO.
SEPT. 8, 1913
APRIL 16, 1914
FENDER WORK
PILES & LUMBER
PILE DRIVING,
DIVERS
E. J. ROURKE
W. H. JONES
LANE QUARRY CO.
E. W. FOLEY CONST. CO.
WILSON & ENGLISH CONST. CO.
DEC. 29, 1910
DEGNON CAPE COD CANAL CONST. CO.
DEC. 29, 1910
HIGHWAY RELOCATION
RAILROAD RELOCATION
NEWARK-MEADOWS CO.
C.W. REYNOLDS (STEAM SHOVEL)
WILSON & ENGLISH CONST. CO. (STEAM SHOVEL)
J. S. PACKARD DREDGING CO.
EASTERN DREDGING CO.
MARYLAND DREDGING CO.
COASTWISE DREDGING CO.
E. W. FOLEY CONST. CO.
STEAM SHOVEL
FEB. 28, 1913
DEC. 15, 1913
STANDARD ENGINEERING CO.
H. K. STOKES, PRES.
OCT. 24, 1914
EXCAVATION
FURST-CLARK CONST. CO.
C. L. CRANDALL
A. D. MORRIS
H. S. ANDREW
ENGINEERS
EXCAVATION
METROPOLITAN DREDGING CO.
R. P. MARSHALL, PRES.
NOV. 19, 1914
DIRECT ON AND AFTER NOV. 1, 1912
EXCAVATION
DEGNON CONTRACTING CO.
JAMES WILSON, ENGINEER
BREAKWATER, RIP-RAP
DEGNON CAPE COD CANAL CONST. CO.
F. D. FISHER, ENG.
BREAKWATER, RIP-RAP, EXCAVATION
MAY 15, 1909
RIP-RAP
DIRECT ON AND AFTER NOV. 1, 1912
BREAKWATER
PENNSYLVANIA STEEL CO.
FEB. 16, 1910
STRAUSS BASCULE & CONCRETE BRIDGE CO.
DESIGNERS & PATENTEES
SUPERSTRUCTURE
JULY 15, 1909
HOLBROOK CABOT & ROLLINS CORP.
SUBSTRUCTURE
FEB. 15, 1910
BUZZARDS BAY RAILROAD BRIDGE
PENNSYLVANIA STEEL CO.
NOV. 30, 1910
SCHERZER ROLLING LIFT BRIDGE CO.
DESIGNERS & PATENTEES
SUPERSTRUCTURE
JULY 21, 1910
HOLBROOK CABOT & ROLLINS CORP.
SUBSTRUCTURE
AUG. 1, 1910
BOURNE HIGHWAY BRIDGE
PENNSYLVANIA STEEL CO.
SUPERSTRUCTURE
APRIL 1, 1912
SCHERZER ROLLING LIFT BRIDGE CO.
DESIGNERS & PATENTEES
SUPERSTRUCTURE
JULY 21, 1911
DRAVO CONTRACTING CO.
APRIL 1, 1912
DEGNON CAPE COD CANAL CONST. CO.
SUBSTRUCTURE
APRIL 1, 1912
SAGAMORE HIGHWAY BRIDGE
CAPE COD CONSTRUCTION COMPANY
WM. BARCLAY PARSONS, CHIEF ENGINEER
H. W. DURHAM, A. S. ACKERMANN, C. T. WARING, RESIDENT ENGINEERS
CONTRACT MARCH 27, 1907, FOR THE WHOLE WORK, PAYMENT TO BE MADE IN BONDS AND STOCKS
WM. BARCLAY PARSONS, CHIEF ENGINEER—EUGENE KLAPP, DEP. CHIEF ENGINEER
BOSTON, CAPE COD & NEW YORK CANAL COMPANY
WM. BARCLAY PARSONS, CHIEF ENGINEER—W. J. DOUGLAS, DEP. CHIEF ENGINEER
CHARTERED JUNE, 1899

FIG. 14.

operation in the development of the mathematical portion. He also acknowledges the constructive criticism of the tidal theory which he received from Mr. Rollin A. Harris, of the U. S. Coast and Geodetic Survey Bureau.

The details for the operation of the canal were worked out by Mr. J. W. Miller, Vice-President of the Canal Company.

Fig. 14 gives a list of the contractors and sub-contractors, with the names of the principal engineers of each and their relations to each other and to the Construction Company and the Canal Company.

PART II.

Hydraulics of the Cape Cod Canal.

The actual physical construction of the canal presents but few items of scientific interest, other than those usually attending new problems in excavating and removing a large quantity of material, or in building breakwaters and bridges. The peculiar value of experience and knowledge gained through the construction of this waterway is in the establishing of new hydraulic data.

Motion of water in open channels due to gravity takes place under three conditions differing in characteristics involving flow: first, when the slope is always in the same direction, as in rivers and canals not subjected to tidal influences; second, when the slope is constantly varying in inclination and direction, the condition developed in the estuaries of rivers, the rise and fall of the tide at one end being the actuating cause; third, when a channel connects two bodies of tidal waters the immediate areas of which are so large, as compared with that of the connecting channel, as to be unaffected by water discharged through it, and having tidal features which differ considerably in range and establishment.

Of the first class, all investigations in the characteristics of flow in rivers are limited by the great irregularity of cross-section, and, therefore, they present but few scientific data as determining the laws of the flow of liquids. It is only in an artificial waterway that sufficient regularity obtains.

The investigations of the flow of water in artificial channels have of necessity been confined almost wholly to channels of comparatively small cross-section, as that in large canals has been reduced to inconsiderable velocity by locks.

An excellent illustration of the second class is the southern end of the Suez Canal between the Little Bitter Lakes and the Red Sea. In the Red Sea the rise and fall of the spring tide is 1.75 m., but the lakes are substantially tideless, the maximum variation from high to low being 0.17 m. The tides are not symmetrical about mean sea level, for the same reason possibly that they are not symmetrical in Buzzards Bay, as will be described later, so that the average maxi-

mum slopes are 0.032 and 0.027 m. per km., northerly and southerly, respectively, the length of uniform canal section being 37.05 km.*

Another illustration is the Pamlico Sound-Beaufort Canal, in North Carolina, where Earl I. Brown, M. Am. Soc. C. E., Major, Corps of Engineers, U. S. A., made some interesting current observations.† This canal has a comparatively small cross-section (90 by 10 ft.), with an average tidal oscillation of 3.8 ft. at Beaufort Harbor. The non-tidal end, however, is greatly affected by the action of the wind in Pamlico Sound.

Of the third class, the Cape Cod Canal is the only large instance of such a channel where all the conditions are conducive to scientific study.

Of the natural waterways of this class, the East River, connecting Long Island Sound and New York Bay, is perhaps the best example. Extensive and careful observations‡ in 1912 by W. M. Black, M. Am. Soc. C. E., Colonel, Corps of Engineers, U. S. A. (now Brig-Gen. and Chief of Engineers), show similar characteristics between the motion of water in that stream and in the Cape Cod Canal, but the irregularity of cross-section of the river renders the making of analytical study of observed results impossible. The Kaiser Wilhelm Canal, popularly known as the Kiel Canal, also connects large bodies of tidal waters, but it is equipped with locks, so that flow through the canal, other than local drainage, is prevented.

Cape Cod Bay and Buzzards Bay, which the canal connects, are large sheets of open water. Cape Cod Bay is really a directly connecting part of the Atlantic Ocean, the opening being about 20 miles across. The bay is circular, with a uniformly curved and substantially unbroken shore line, and with considerable depth of water, ranging from 12 to 25 fathoms, over a flat floor with no abrupt hollows or ridges. Buzzards Bay, on the other hand, is much longer than it is wide, being 50 miles long and having an average width of about 7 miles, with an average depth of about 10 fathoms at its opening, shoaling to about 2 fathoms at the upper end, except in the original narrow channels to Monument Beach and Wareham, or in local shoals, where the depth is still less.

* The facts regarding the tides and currents are contained in the reports of the "Commission Consultative Internationale des Travaux" for 1906 and 1907.

† The results are published in the March–April, 1912, number of *Professional Memoirs.*

‡ *Professional Memoirs,* June, 1913.

These bays are affected by two different tidal waves, which are quite dissimilar in their establishments, ranges, and other characteristics. The wave that passes up Buzzards Bay is a branch of the great Atlantic wave which runs northward along the coast, its velocity of travel being somewhat retarded in its passage over the comparatively shoal water along the coast. The wave in Cape Cod Bay is another branch of the same great wave, which, traveling faster in the deeper water off shore, strikes the Nova Scotia coast and thence is deflected in part westward and then southward. These two waves meet and interfere in Vineyard Sound, and, in the irregularity produced by such interference, add to the difficulties of navigation and cause fogs owing to their difference in temperature. The mean range of the tide in Cape Cod Bay is 9.0 ft. at Boston, 9.6 ft. at Plymouth, 8.9 ft. at the entrance of the canal,* and 9.2 ft. at Provincetown, whereas the mean range in Buzzards Bay is 4.0 ft. at New Bedford and 3.6* ft. at the entrance to the canal.

Briefly, then, the problem developed by the construction of the Cape Cod Canal is an extremely complex one in hydrodynamics, being the analysis of the motion of water in a canal of considerable magnitude connecting two seas, the tides in which differ to a great extent, both as to phase and amplitudes.

Before entering on the analysis of the problem, the basal facts, as determined by the records, should be set forth.

In the early days, when a canal was under consideration, the known variation in tidal head at the two ends of the canal precluded any consideration of a sea-level canal through an unanalyzed fear of disastrous results, and it was not until 1861 that any study was made of the tidal phenomena and conditions, when a legislative committee undertook the investigation. As explained previously, the committee called to its aid Gen. Totten, Professor Bache, and Commander Davis, and these gentlemen in turn placed Mr. Henry Mitchell in charge of the tidal observations. Mr. Mitchell's reports are set forth in detail in the general report of the committee, published as a State document in 1864. He carried these observations over one month, therefore covering a lunar cycle, and found that the mean rise and fall was 9.17 and 4.11 ft. at the east and west ends, respectively, results that agree closely with those determined by observations

* From records obtained subsequent to the opening of the canal.

extending over some years, namely, 9.02* and 3.91* ft.; but his accuracy ended there, because, knowing that the two tidal waves oscillated uniformly above and below mean sea level at points along the coast, according to his own and other determinations for the coast survey, he assumed that the same conditions obtained in the locality under investigation. Apparently, he set up two tide recording stations, where he ascertained the heights of low and high waters and computed mean sea level in each case as half way between mean high and mean low. He omitted to run a line of check levels across the isthmus to determine whether his computed mean sea levels were actually, as he assumed them to be, the same datum plane—another illustration of the danger of making assumptions in engineering work without actual knowledge. Therefore, he determined that though the durations of the flood and ebb tides in Cape Cod Bay were the same, in Buzzards Bay the flood endured for 7 hours 01 min. and the ebb for 5 hours 23 min., and, as a corollary, that the maximum differences in simultaneous tidal heights between the two ends on the flood and on the ebb were not nearly equal. As indicative of the lack of knowledge then existing regarding the flow of water in large open channels, the tidal report contained a conclusion, which the Board thought of sufficient importance to put in Italics, reading: "The greater the transverse section of the free canal, the more rapid will be the flow through it." It seems extraordinary that as recently as 1860 that statement was sufficiently novel to be deemed, by men like Professor Bache and Mr. Henry Mitchell, as "interesting" and worthy of being put in Italics in a Government report.

Tidal observations were begun by the writer in October, 1907, when automatic recording mareograph instruments, of the U. S. Coast Survey type, were set up at Monument Beach in Buzzards Bay and at Barnstable in Cape Cod Bay. When the canal excavation began, these instruments were transferred to the points just within the canal at each end (Canal Stations 35 and 380), where they have been maintained continuously under observation. The differences in tidal heights and times between the first selected points and those within the canal were found to be so small as to be negligible. Accurate levels were run between the two instruments, so that their

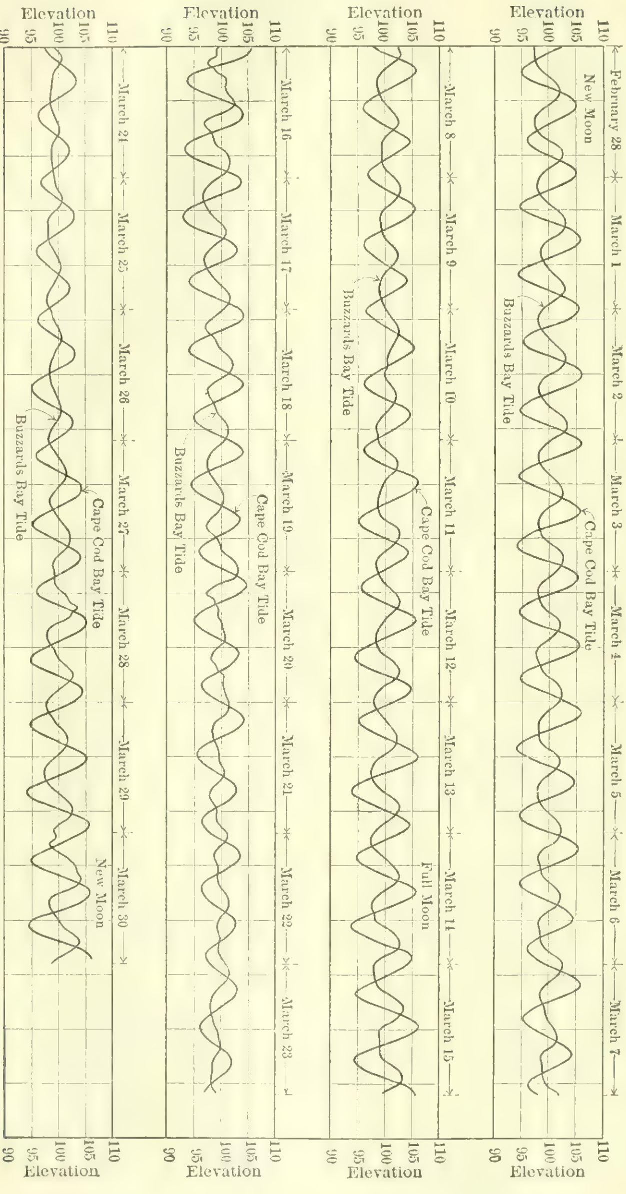
* From records obtained previous to the opening of the canal.

readings are based on a common datum, which, in order to avoid "minus" elevations, was taken as 100 ft. below mean sea level.

The simultaneous elevations of the two tides for a lunar cycle are shown superimposed in Fig. 15. From this diagram it will be seen that the tides in Cape Cod Bay are quite symmetrical above and below mean sea level and the path of the curve is regular, and closely approximates a sine curve. This is to be expected, because the smooth contours and depth of water permit the tidal wave to pass without sensible retardation or uneven interruption. In Buzzards Bay the case is quite different, for the depth of water, especially at the upper end, is so slight that the difference in depth between high and low water is considerable, being nearly 4 ft. on an average depth of about 10 ft., a variation of nearly 40 per cent. The consequence is that though the tide flows in freely, in ebbing it soon reaches a point where bottom friction accentuated by great bottom unevenness, produces a serious effect and retards the outward flow. This is shown distinctly in Fig. 15 where the loop of the tide curve above mean sea level is regular and the trough or lower loop of the tide curve is distinctly irregular. It will be noted, however, that the periods of flood and ebb are equal in duration, and not unequal, as Mr. Mitchell supposed. In Buzzards Bay mean high tide is 2.14 ft. above mean sea level and mean low tide 1.45 ft. below it, but if the lower branch of the Buzzards Bay curve is projected, as shown by the dotted line in Fig. 16—a typical marcograph record—the curve becomes regular, and the heights of high and low water are equal above and below mean sea level.

It was found that the difference in time between high water at the two ends is 3 hours 15 min., or substantially the same as determined by Mr. Mitchell, so that, for all practical purposes, the tides are just half tide apart.

In Cape Cod Bay the elevation of mean high water is 104.42 ft., datum plane being taken at 100 ft. below mean sea level, and the elevation of mean low water is 95.50 ft. In Buzzards Bay the elevations are 102.14 and 98.55 ft., respectively. Fig. 16 shows the curves of two mean tides, and from this diagram it will be seen that the maximum instantaneous differences in elevation of the water surfaces occur 30 min. after high water and 30 min. after low water in Cape



MAREOGRAPH FOR A WHOLE LUNAR CYCLE, FEBRUARY 28TH, TO MARCH 30TH, 1911

FIG. 15.

Cod Bay, and that for 1 hour 30 min. in each case the difference in elevation is practically unchanged, the paths of the two curves being nearly parallel. It is fortunate, from the scientific standpoint, that the incompleteness or irregularity of the trough of the Buzzards Bay tide curve does not affect the maximum differences, as the irregularity does not begin until after the instantaneous difference has begun to diminish sensibly. The irregularity, therefore, affects only hydrodynamics and flow conditions that are less than the maximum.

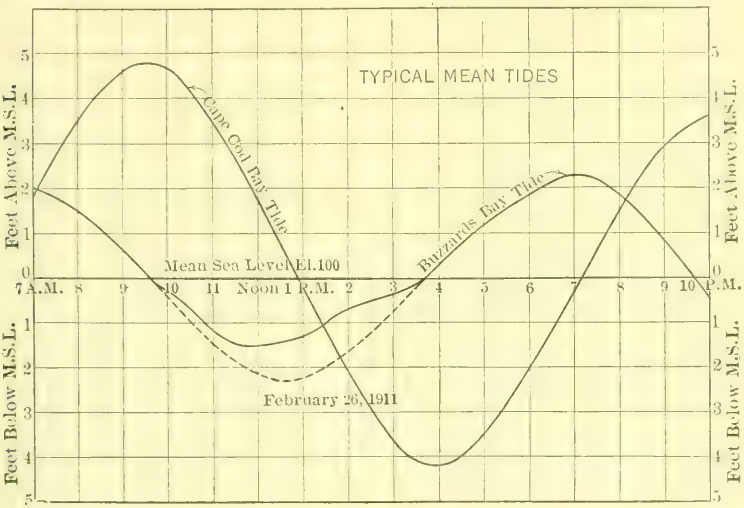


FIG. 16.

As the slope remains practically constant for a time long enough to eliminate variations produced by wave propagation, the canal becomes an example of motion of water affected by variations in elevations at both ends, and also of a large channel in which the "head" is constant.

In actuality, the tidal elevations vary considerably from the mean figures, according to the moon phases producing "spring" and "neap" tides and also to irregularities following wind action. Tables 2 to 9, inclusive, give the numbers of tides in each year grouped in variations of foot differences, from 1908 to 1915, inclusive. The average maximum differences, as determined from the data contained in these tables, are 5 ft. producing easterly and 5.3 ft. producing westerly currents, including the effects of storms. The maximum difference occurred on January 13th, 1915, when it was 9.5 ft. westerly, being

TABLE 2.—RECORD OF TIDAL DIFFERENCES, CAPE COD CANAL, FOR 1908.

Maximum Tidal Differences, in Feet.

	JAN.		FEB.		MAR.		APR.		MAY.		JUNE.		JULY.		AUG.		SEPT.		OCT.		NOV.		DEC.		1908	
	+	−	+	−	+	−	+	−	+	−	+	−	+	−	+	−	+	−	+	−	+	−	+	−	+	−
Maximum.....	8.6	8.5	9.2	7.6	8.0	8.8	7.0	8.5	7.1	7.2	7.0	7.5	7.5	8.1	7.0	8.0	6.8	8.0	8.4	8.2	7.6	7.3	7.3	7.4	9.2	8.8
Minimum.....	1.3	3.5	3.3	4.4	3.2	2.8	2.9	3.3	3.2	3.7	3.4	3.4	3.8	4.0	3.6	3.6	2.5	3.3	2.1	3.8	2.6	2.9	3.2	3.5	1.3	2.8

East Current +
West " —

MAX. DIFF.	NUMBER OF TIDES.																									
8 +	2	1	4	0	2	6	0	2	0	0	0	0	0	2	0	1	0	2	4	1	0	0	0	0	12	15
7 +	0	9	9	4	8	8	1	6	1	3	1	5	5	4	2	7	0	7	6	11	1	8	1	4	33	76
6 +	12	15	6	19	7	19	4	16	9	20	11	11	7	13	6	8	11	15	8	10	10	13	10	17	101	176
5 +	14	21	20	28	13	16	21	19	25	17	22	26	26	22	6	23	12	13	17	23	18	17	16	24	210	249
4 +	23	11	13	5	21	6	19	10	19	13	18	9	21	19	33	19	16	17	18	14	10	15	21	12	232	150
3 +	7	3	4	0	9	4	11	5	6	6	6	7	3	0	13	2	17	4	5	1	16	2	11	3	108	37
2 +	0	0	0	0	0	1	2	0	0	0	0	0	0	0	0	0	2	0	2	0	3	3	0	0	9	4
1 +	2	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	0
	60	60	56	56	60	60	58	58	60	59	58	58	60	60	60	60	58	58	60	60	58	58	59	60	707	707

TABLE 3.—Record of Tidal Differences, Cape Cod Canal, for 1909.

Maximum Tidal Differences, in Feet.

	Jan.		Feb.		Mar.		Apr.		May.		June.		July.		Aug.		Sept.		Oct.		Nov.		Dec.		1909.	
	+	−	+	−	+	−	+	−	+	−	+	−	+	−	+	−	+	−	+	−	+	−	+	−	+	−
Maximum.....	6.8	8.0	6.8	7.8	5.8	7.7	6.0	6.3	5.6	6.7	5.6	5.4	5.5	5.4	7.8	7.4	8.2	8.2	8.1	8.3	7.8	9.2	6.8	8.3	8.2	9.2
Minimum......	2.2	3.3	2.5	2.5	1.7	3.3	2.0	2.8	2.9	2.3	3.4	2.6	3.2	2.3	2.6	3.4	3.8	3.0	2.7	3.3	2.9	3.8	3.2	4.2	1.7	2.3

East Current +
West " —

Max. Diff.	Number of Tides.																									
8 +	0	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	3	1	1	2	0	6	0	2	4	12
7 +	0	2	0	5	0	6	0	0	0	0	0	0	0	0	2	1	6	8	6	9	5	5	0	8	19	44
6 +	1	12	1	11	0	5	1	3	0	3	0	0	0	0	4	4	11	7	10	8	9	7	9	21	46	81
5 +	12	23	14	16	10	22	11	14	5	9	7	4	14	5	14	18	20	24	24	23	18	17	23	19	172	194
4 +	22	16	16	19	19	20	25	24	31	27	30	36	28	28	35	25	17	14	13	15	17	21	25	10	278	255
3 +	18	6	19	2	28	7	18	14	23	18	21	17	18	23	4	12	1	3	5	3	8	2	3	0	166	107
2 +	7	0	4	1	2	0	3	3	1	3	0	1	0	4	1	0	0	0	1	0	1	0	0	0	20	12
1 +	0	0	0	0	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	0
	60	60	54	54	60	60	58	58	60	60	58	58	60	60	60	60	58	57	60	60	58	58	60	60	706	705

TABLE 4.—Record of Tidal Differences, Cape Cod Canal, for 1910.

Maximum Tidal Differences, in Feet.

	Jan.		Feb.		Mar.		Apr.		May.		June.		July.		Aug.		Sept.		Oct.		Nov.		Dec.		1910.	
	+	−	+	−	+	−	+	−	+	−	+	−	+	−	+	−	+	−	+	−	+	−	+	−	+	−
Maximum.....	7.2	7.1	7.8	7.2	7.5	7.1	8.0	7.6	8.5	7.6	8.2	7.3	7.9	7.0	7.8	7.0	8.2	7.1	8.8	8.2	8.7	8.5	8.2	8.5	8.8	8.5
Minimum......	1.3	3.1	2.2	1.7	3.3	2.3	3.4	2.4	3.7	3.8	4.2	4.0	4.1	3.6	3.8	3.2	3.4	2.9	2.8	3.1	3.1	3.4	3.6	3.0	1.3	1.7

East Current +
West " —

Number of Tides.

Max. Diff.																										
8 +	0	0	0	0	0	0	1	0	4	0	1	0	0	0	0	0	3	0	6	1	3	3	3	1	21	5
7 +	1	3	3	4	2	4	4	8	8	8	4	3	10	1	10	2	9	7	6	9	7	8	5	4	69	61
6 +	17	10	12	14	8	8	12	6	6	7	16	13	25	19	19	18	19	16	15	13	11	14	9	16	169	154
5 +	17	27	16	15	22	14	15	15	24	21	22	14	15	25	19	25	14	18	18	18	19	17	20	24	221	233
4 +	13	10	16	13	2	18	17	13	16	20	15	28	10	12	11	11	8	11	7	13	16	11	21	8	170	168
3 +	9	10	5	5	8	13	9	14	2	4	0	0	0	3	1	4	5	5	7	6	2	5	2	7	50	76
2 +	2	0	2	2	0	3	0	2	0	0	0	0	0	0	0	0	0	1	1	0	0	0	0	0	5	8
1 +	1	0	0	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	1
	60	60	54	54	60	60	58	58	60	60	58	58	60	60	60	60	58	58	60	60	58	58	60	60	706	706

TABLE 5.—Record of Tidal Differences, Cape Cod Canal, for 1911.

Maximum Tidal Differences, in Feet.

	Jan.		Feb.		Mar.		Apr.		May.		June.		July.		Aug.		Sept.		Oct.		Nov.		Dec.		1911.	
	+	—	+	—	+	—	+	—	+	—	+	—	+	—	+	—	+	—	+	—	+	—	+	—	+	—
Maximum.....	7.7	7.7	8.9	7.5	6.9	7.2	7.1	7.7	7.2	7.4	6.6	7.4	7.3	7.3	6.8	7.5	6.8	7.1	7.0	7.2	7.8	7.2	7.8	7.9	8.9	7.9
Minimum......	3.5	2.8	2.1	2.3	2.3	3.2	2.2	3.3	3.2	3.6	3.3	4.3	3.7	4.2	3.4	3.5	1.6	3.1	1.8	2.8	2.4	3.4	2.7	4.0	1.6	2.3

East Current +
West " —

Max. Diff. — Number of Tides.

Max. Diff.	Jan. +	Jan. —	Feb. +	Feb. —	Mar. +	Mar. —	Apr. +	Apr. —	May +	May —	June +	June —	July +	July —	Aug. +	Aug. —	Sept. +	Sept. —	Oct. +	Oct. —	Nov. +	Nov. —	Dec. +	Dec. —	1911 +	1911 —
8 +	0	0	6	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	6	0
7 +	5	5	8	1	0	3	1	4	2	5	0	4	2	2	0	3	0	2	2	3	2	2	1	3	26	37
6 +	13	17	8	6	13	24	10	14	10	12	5	12	9	11	11	11	12	19	6	18	7	14	4	13	108	171
5 +	21	23	7	18	21	15	17	20	11	19	16	29	16	32	25	27	16	17	17	18	20	18	18	26	205	262
4 +	15	10	9	20	12	14	18	13	23	21	26	13	31	15	18	15	17	11	17	11	16	19	24	18	226	180
3 +	6	4	15	6	10	4	9	7	14	3	11	0	2	0	6	4	12	9	13	8	11	5	9	0	118	50
2 +	0	1	1	3	4	0	3	0	0	0	0	0	0	0	0	0	0	0	4	2	2	0	1	0	15	6
1 +	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	0	1	0	0	0	0	0	2	0
	60	60	54	54	60	60	58	58	60	60	58	58	60	60	60	60	58	58	60	60	58	58	60	60	706	706

TABLE 6.—RECORD OF TIDAL DIFFERENCES, CAPE COD CANAL, FOR 1912.

Maximum Tidal Differences, in Feet.

	JAN.		FEB.		MAR.		APR.		MAY.		JUNE.		JULY.		AUG.		SEPT.		OCT.		NOV.		DEC.		1912.	
	+	−	+	−	+	−	+	−	+	−	+	−	+	−	+	−	+	−	+	−	+	−	+	−	+	−
Maximum	7.4	8.7	7.2	8.0	8.0	8.2	6.4	7.5	6.9	8.1	7.7	7.3	8.1	7.6	7.9	7.9	8.9	8.5	8.5	7.5	7.8	7.6	7.6	7.4	8.9	8.7
Minimum......	1.9	3.7	2.3	3.5	2.5	3.3	2.6	3.1	2.1	3.1	2.9	3.9	3.6	1.9	3.7	3.8	2.2	2.9	2.9	3.1	2.9	2.6	2.9	2.5	1.9	1.9

East Current +
West " —

MAX. DIFF. NUMBER OF TIDES.

8 +	0	3	0	2	1	2	0	0	0	1	0	0	1	0	0	0	2	2	2	0	0	0	0	0	6	10
7 +	4	2	2	2	7	4	0	3	0	1	2	4	2	4	7	8	5	7	2	4	6	3	3	2	40	44
6 +	7	11	8	14	8	25	3	14	10	14	6	16	9	11	7	9	9	14	9	15	11	11	8	7	95	161
5 +	9	30	15	20	19	12	22	22	25	25	24	19	16	21	17	28	8	20	17	17	19	17	25	23	216	254
4 +	14	13	18	15	14	11	20	15	15	13	22	17	28	21	26	12	17	9	22	14	16	20	16	18	228	178
3 +	21	1	12	3	8	6	10	4	7	6	3	1	4	2	3	3	14	5	7	10	5	6	6	9	100	56
2 +	4	0	1	0	3	0	3	0	3	0	1	0	0	1	0	0	3	1	1	0	1	1	1	1	21	4
1 +	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	0
	60	60	56	56	60	60	58	58	60	60	58	57	60	60	60	60	58	58	60	60	58	58	59	60	707	707

TABLE 7.—Record of Tidal Differences, Cape Cod Canal, for 1913.

Maximum Tidal Differences, in Feet.

	Jan.		Feb.		Mar.		Apr.		May.		June.		July.		Aug.		Sept.		Oct.		Nov.		Dec.		1913.	
	+	—	+	—	+	—	+	—	+	—	+	—	+	—	+	—	+	—	+	—	+	—	+	—	+	—
Maximum.....	8.2	7.8	8.2	8.3	7.9	8.0	6.8	7.6	6.6	7.1	6.2	7.2	7.1	6.8	7.0	7.5	7.7	7.9	7.2	8.0	6.9	7.5	7.1	7.3	8.2	8.3
Minimum......	3.1	2.6	3.4	3.5	2.6	2.5	2.4	3.4	1.9	3.6	3.4	3.6	3.8	3.6	3.6	3.5	3.2	3.5	2.9	3.4	2.7	3.6	2.0	3.6	1.9	2.5
East Current +																										
West " —																										
Max. Diff.	Number of Tides.																									
8 +	1	0	1	1	0	1	0	0	0	0	0	0	0	0	0	0	0	0	0	1	0	0	0	0	2	3
7 +	6	6	2	9	7	3	0	5	0	3	0	2	1	0	1	5	7	7	3	13	0	2	1	1	28	56
6 +	8	10	8	7	6	10	8	12	4	12	4	10	8	13	12	10	7	11	9	14	6	13	3	17	83	139
5 +	18	17	14	10	12	20	15	18	12	24	25	27	25	23	22	21	18	18	10	10	17	22	26	26	214	236
4 +	23	18	20	22	21	13	19	17	28	18	24	16	21	22	23	20	18	15	26	15	22	17	21	12	266	205
3 +	4	7	9	5	12	11	14	6	13	3	5	3	5	2	2	4	8	7	11	6	10	4	7	4	100	62
2 +	0	2	0	0	2	2	2	0	2	0	0	0	0	0	0	0	0	0	1	0	3	0	2	0	12	4
1 +	0	0	0	0	0	0	0	0	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	0
	60	60	54	54	60	60	58	58	60	60	58	58	60	60	60	60	58	58	60	59	58	58	60	60	706	705

TABLE 8.—Record of Tidal Differences, Cape Cod Canal, for 1914.

Maximum Tidal Differences, in Feet.

	Jan.		Feb.		Mar.		Apr.		May.		June.		July.		Aug.		Sept.		Oct.		Nov.		Dec.		1914.	
	+	−	+	−	+	−	+	−	+	−	+	−	+	−	+	−	+	−	+	−	+	−	+	−	+	−
Maximum.....	7.0	7.7	8.1	8.3	7.1	8.4	7.1	7.6	7.7	8.3	7.5	7.7	6.5	7.0	6.3	6.3	7.8	7.0	7.0	7.0	7.5	7.1	7.0	7.6	8.1	8.4
Minimum......	3.3	2.5	2.7	2.9	2.3	3.6	2.2	2.8	3.2	3.2	3.8	3.8	3.2	3.4	2.9	2.9	2.1	3.0	2.5	3.1	2.8	2.8	2.5	2.6	2.1	2.5

East Current +
West " −

Max. Diff.	Number of Tides.																									
8 +	0	0	1	1	0	4	0	0	0	2	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	7
7 +	2	5	2	1	1	9	3	2	2	6	2	3	0	1	0	0	1	1	3	3	3	2	2	2	21	33
6 +	12	11	6	13	9	5	4	8	10	9	7	14	3	9	3	9	6	14	6	9	2	4	7	10	75	115
5 +	24	23	14	19	17	22	14	19	13	21	22	22	21	31	18	25	17	22	9	17	13	27	21	17	203	268
4 +	18	11	13	13	20	17	19	21	22	18	24	16	23	12	20	17	17	13	22	22	23	20	16	20	237	200
3 +	4	9	14	6	8	3	15	7	13	4	3	3	12	4	18	8	13	8	17	9	15	4	11	10	143	115
2 +	0	3	4	1	5	0	3	1	0	0	0	0	0	0	1	1	4	0	3	0	2	1	3	1	25	8
1 +	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
	60	60	54	54	60	60	58	58	60	60	58	58	59	60	60	60	58	58	60	60	58	58	60	60	705	706

TABLE 9.—RECORD OF TIDAL DIFFERENCES, CAPE COD CANAL, FOR 1915.

Maximum Tidal Differences, in Feet.

	Jan.		Feb.		Mar.		Apr.		May.		June.		July.		Aug.		Sept.		Oct.		Nov.		Dec.		1915.	
	+	−	+	−	+	−	+	−	+	−	+	−	+	−	+	−	+	−	+	−	+	−	+	−	+	−
Maximum.....	8.0	9.5*	6.1	7.5	6.5	7.1	7.6	7.1	7.8	6.6	8.0	6.3	7.0	6.5	6.7	6.6	7.0	6.2	6.0	6.2	8.3	6.6	7.5	7.2	8.3	9.5*
Minimum......	2.9	2.4	2.5	2.5	2.6	2.2	2.0	2.0	3.2	2.9	3.9	3.4	3.7	2.9	2.6	2.7	2.7	2.6	2.1	2.4	2.8	1.7	3.0	3.0	2.0	1.7

East Current +
West " —

NUMBER OF TIDES.

Max. Diff.	Jan. +	Jan. −	Feb. +	Feb. −	Mar. +	Mar. −	Apr. +	Apr. −	May +	May −	June +	June −	July +	July −	Aug. +	Aug. −	Sept. +	Sept. −	Oct. +	Oct. −	Nov. +	Nov. −	Dec. +	Dec. −	1915 +	1915 −
8 +	1	1	0	0	0	0	0	0	0	0	1	0	0	0	0	0	0	0	0	0	2	0	0	0	4	1
7 +	5	0	0	1	0	2	5	2	2	0	2	0	1	0	0	0	5	0	0	0	6	0	6	1	32	6
6 +	7	2	3	13	6	7	4	6	14	5	6	2	5	6	6	1	7	4	1	2	5	9	5	2	69	59
5 +	18	15	17	20	17	18	24	21	21	13	20	9	22	19	25	23	19	20	18	15	20	9	11	10	235	192
4 +	21	31	16	12	17	17	22	14	16	30	28	30	27	27	20	20	13	18	23	19	12	23	24	32	239	273
3 +	7	8	14	5	13	12	2	9	4	10	1	17	5	7	7	11	13	13	13	18	12	16	13	15	104	144
2 +	1	2	4	4	7	3	1	6	0	2	0	0	0	1	2	2	1	3	5	6	1	0	0	0	22	29
1 +	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	0	0	0	1
	60	59	54	55	60	59	58	58	60	60	58	58	60	60	60	60	58	58	60	60	58	58	59	60	705	705

* Storm—Jan. 13th, 1915.

produced by a great tide on the Massachusetts coast, the result of a violent storm. The minimum difference was 1 ft. 3 in. easterly. It will be seen, on study of these tables, that the excessive tides are comparatively few, the greater majority, about 83.5% in fact, giving heads that vary from 3 to 6 ft., and only about 2% giving differences

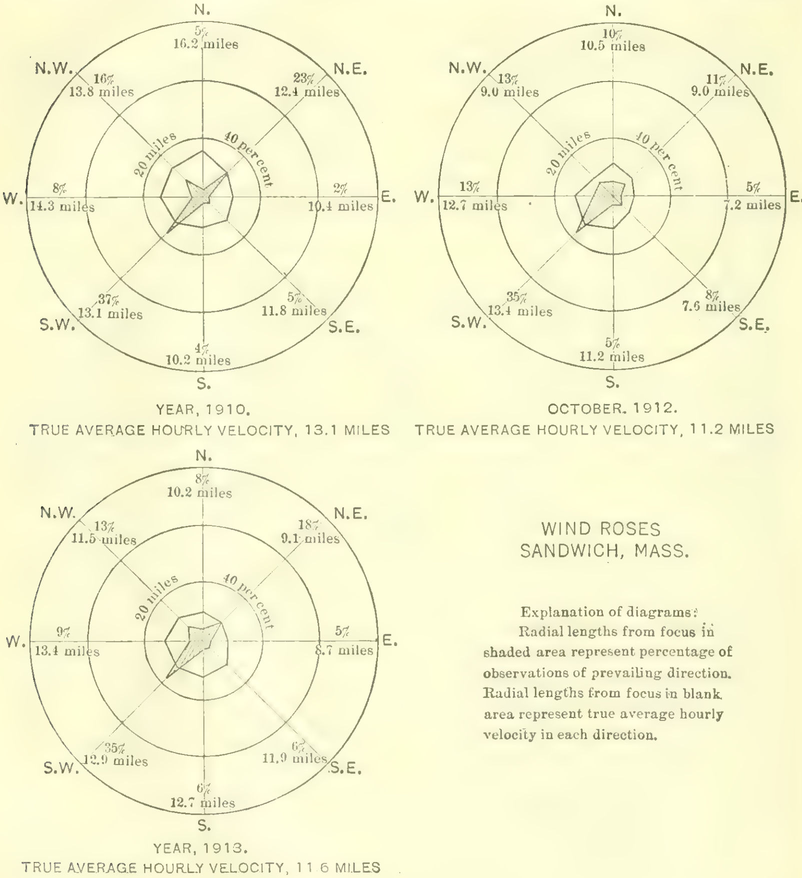


FIG. 17.

exceeding 7 ft. In these extreme cases the maximum difference lasts for a few minutes only, as the tidal curves no longer maintain their parallelism.

The difference in elevation at any instant is the hydraulic head that produces the velocity of the current, and as it occurs in a length of canal of 34 500 ft. (the distance apart of the automatic tide gauges),

the resulting mean maximum slope is 0.000154. It is interesting to point out that this slope is nearly four times as steep as the maximum slope that would have obtained at the Panama Canal had it been built at sea level, taking the maximum tidal oscillation at Panama at 22 ft., only one-half of which would have been above or below mean sea level, and neglecting the negligibly small tidal difference at Colon.

In order to ascertain the action of the wind, a recording anemometer was established at the same time as the tide gauges. Monthly wind roses for selected typical years are shown in Fig. 17, which are self-explanatory. Table 10 gives a record of winds having a velocity which exceeded 40 miles per hour between 1908 and 1915, inclusive. The prevailing winds are northeast and southwest; they blow across the canal, and produce no direct effect.

TABLE 10.—True Wind Velocities of More than 40 Miles per Hour, from 1908 to December, 1915.

Date.	Time.	Total movement, in miles.	Duration, in hours.	Maximum velocity, in miles per hour.	Mean velocity, in miles per hour.	Direction.
Jan. 24, 1908...	11.00 A. M.—12.00 M.	40	1	40.8	40.0	N. E.
Dec. 26, 1909...	3.30 A. M.— 9.30 P. M.	799	18	55.2	44.4	N. W.
Jan. 14, 1910...	6.15 P. M.—10.15 P. M.	160	4	42.2	40.0	N. E.
Jan. 15, 1910...	6.00 A. M.— 4.30 P. M.	436	10.5	45.1	41.5	N.
Feb. 1, 1910...	8.45 A. M.— 1.00 P. M.	193	4.25	41.5	40.8	N. W.
Feb. 4, 1910...	10.00 A. M.— 3.00 P. M.	215	5	45.1	43.0	N.
Feb. 18, 1910...	1.30 A. M.— 2.45 P. M.	52	1.25	43.7	41.5	N. W.
Nov. 27, 1910...	7.15 A. M.—10.15 A. M.	122	3	40.8	40.8	N. W.
Mar. 16, 1911...	7.00 A. M.— 9.30 A. M.	100	2.5	43.0	40.0	W
July 29, 1911...	3.30 P. M.— 5.30 P. M.	84	2	45.1	42.2	S. W.
Dec. 28, 1911...	1.15 P. M.— 2.15 P. M.	40	1	45.1	40.0	W.
Feb. 22, 1912...	9.00 A. M.— 1.30 P. M.	190	4.5	48.0	42.2	S. W.
Mar. 15, 1912...	8.30 P. M.— 9.30 P. M.	40	1	45.1	40.0	W.
Jan. 3, 1913...	10.30 P. M.—10.45 P. M.	13	0.25	42.2	42.2	S. E.
Jan. 4, 1913...	10.45 P. M.— 3.00 A. M.	222	4.25	45.1	40.0	S. W.
Mar. 24, 1913...	1.10 P. M.— 3.00 P. M.	93	1.83	42.2	41.3	S. W.
Mar. 27, 1913...	7.45 A. M.— 4.40 P. M.	443	8.9	46.6	49.5	S.
Jan. 12, 1914...	4.15 P. M.— 5.25 P. M.	59	1.16	45.1	41.5	S. W.
Jan. 13, 1914...	1.00 A. M.— 1.15 A. M	13	0.25	41.5	41.5	N. W.
Mar. 1, 1914...	3.45 P. M.— 4.30 P. M.	36	0.75	48.0	47.3	S. E.
Dec. 13, 1914...	11.35 P. M.—12.00 M.	23	0.58	40.8	40.0	S. E.
Dec. 14, 1914...	12.00 M. — 2.05 A. M.	85	2.08	42.2	40.8	N. E.
Nov. 5, 1915...	8.05 P. M.— 8.30 P. M.	17	0.41	40.8	40.8	N. W.
Dec. 13, 1915...	5.00 P. M.— 7.30 P. M.	113	2.50	50.0	45.0	N. E.

In order to determine whether any appreciable difference in tidal conditions was produced by the flow of water through the canal, the tidal records were taken for four calendar months prior to the opening of the canal and during the same months after the opening. An average of all the tides in this period is given in Table 11.

TABLE 11.—ELEVATIONS OF MEAN HIGH AND MEAN LOW WATER AT BOTH ENDS OF THE CANAL BEFORE AND AFTER OPENING TO FLOW.

	Buzzards Bay.		Cape Cod Bay.	
	High water.	Low water.	High water.	Low water.
Before opening	102.17	98.26	104.56	95.54
After opening	102.14	98.55	104.42	95.50

It will be seen that no appreciable influence was produced on the tidal elevations in Cape Cod Bay, nor on high-water elevation in Buzzards Bay, but that the elevation of low water in Buzzards Bay was raised 0.29 ft., or about 3½ in., due undoubtedly to the inability of the water to discharge itself freely in the shallow depth existing at low tide.

In order to ascertain all the conditions affecting, or produced by, the motion of water in the canal, an elaborate system of taking measurements was organized after the excavation had been completed so as to give free flow.

Observation posts were established at Stations 45, 80, 125, 172, 225, 275, 325, 375, 399+50, and 410. Station 45 is where the canal has a bottom width of 200 ft. At Station 80 the bottom width had been reduced to the normal 100 ft., the narrowing commencing at Station 70, Station 80 being selected for observation as being about the first or last place, according to direction of current, where the flow was believed to be normal for a narrow section. Through the 100-ft. section the observing stations were at about 5 000-ft. intervals to Station 399+50, where the cross-section is increased to 250 ft. bottom width. Station 410 is the end of the canal. At all these stations, tide boards were set up, and their elevations were carefully checked.

At each post there was an experienced observer with two assistants, and the observations were made simultaneously and continuously for nearly 15 hours, so as to cover fully a complete tidal cycle.

One of the assistants was in a boat supplied with floats, of which he placed one in the center of the canal every 15 min. about 600 ft. above (according to current) the observer. The second assistant was stationed 500 ft. above the observer sighting across the canal over marks set at right angles to the line of the canal. The accurately placed canal lights served well for such purpose. When the float passed the assistant he signaled to the observer who recorded the time of passage, and again the time when the float passed similar marks at the observing station. These times by reduction gave the center surface velocities. At 15-min. intervals the observer also recorded the elevation of the water, as shown by the tide boards, and noted the direction, time of stopping, and reversing of the current, action of the wind, passing of boats, and other circumstances affecting the flow. The observers' watches were synchronized by the engineer in charge. To eliminate errors these observations were repeated on July 26th and August 26th, 1914, being selected as days when a head differential greater than the mean was to be expected. All reductions and computations were made in the Chief Engineer's office from original notebooks, the observers being given no opportunity to check their recorded observations with their own figures or those of adjacent observers.

To permit these records to be visualized, they have been plotted in a series of diagrams.

The first of the series, Fig. 18, shows the simultaneous elevations of the water surface as taken at several observation stations. By connecting the elevations at the same time points by straight lines between observation stations, instantaneous profiles of water surface are obtained. These lines are not straight from one end of the canal to the other, but are substantially straight only for those portions of the canal where the cross-section is uniform, that is, between Stations 80 and 375, where the canal has a bottom width of 100 ft. The lines between Stations 45 and 80 are flatter on account of the greatly increased cross-section of the canal. The slight irregularities occurring between Stations 375 and 380 are due to the contraction of the Buzzards Bay Bridge.

If curves are drawn osculatory to these lines, they will give the elevations of high water and low water at all points through the canal. The greater the number of simultaneous water profiles, the greater

will be the number of points given on the curves, but the more accurately determined loci of the curves will not vary appreciably from the curves as drawn on Figs. 18 and 19. These curves show that the elevations of high and low water are not on straight lines between the two ends of the canal, as the surface slopes are, but lie on pronounced curves; and that, for substantial portions of the length of the canal, tides rise neither as high nor fall as low even as the high and low minimum of Buzzards Bay. This result was not anticipated, nor was it suggested by any writer on the expected tidal results. The differences are quite material, as shown by Table 12, which gives the actual elevations at points in the canal and computed elevations if the high and low water were on straight lines, connecting the high and low points at Buzzards Bay and Cape Cod Bay.

TABLE 12.—Deviation of Actual High-Water and Low-Water Lines from Straight Lines Connecting the Extreme Elevations at the Two Ends of the Canal.

Observations of July 26th, 1916.

Stations.	High Water.			Low Water.		
	Actual elevation.	Elevation of straight line.	Difference, in feet.	Actual elevation.	Elevation of straight line.	Difference in feet.
35 + 00	104.80	104.8	......	93.87	93.87	....
45 + 00	104.96	104.77	+ 0.19	93.87	93.98	+ 0.11
80 + 00	104.60	104.66	— 0.06	94.23	94.40	+ 0.17
125 + 00	104.16	104.52	— 0.36	94.70	94.93	+ 0.23
172 + 00	103.50	104.37	— 0.87	95.70	95.48	— 0.22
225 + 00	102.80	104.20	— 1.40	96.90	96.09	— 0.81
275 + 00	102.33	104.04	— 1.71	97.60	96.67	— 0.93
325 + 00	102.75	103.88	— 1.13	98.16	97.25	— 0.91
374 + 20	103.25	103.70	— 0.45	98.21	97.80	— 0.59
380 + 00	103.33	103.67	— 0.34	98.16	97.87	— 0.29
399 + 50	103.66	103.66	....	98.00	98.00	....

Observations of August 26th, 1916.

Stations.	Actual elevation.	Elevation of straight line.	Difference, in feet.	Actual elevation.	Elevation of straight line.	Difference in feet.
35 + 00	105.58	105.58	....	94.75	94.75	....
45 + 00	105.62	105.53	+ 0.09	94.72	94.84	+ 0.12
80 + 00	105.50	105.30	+ 0.20	94.50	95.16	+ 0.66
125 + 00	104.80	105.02	— 0.22	95.70	95.59	— 0.11
172 + 00	104.13	104.73	— 0.60	96.20	96.05	— 0.15
225 + 00	103.30	104.46	— 1.16	96.90	96.45	— 0.45
275 + 00	102.50	104.15	— 1.65	97.55	96.94	— 0.61
325 + 00	102.50	103.85	— 1.35	98.30	97.42	— 0.88
375 + 00	103.04	103.53	— 0.49	98.65	97.90	— 0.75
380 + 00	103.10	103.50	— 0.40	98.60	97.95	— 0.65
410 + 00	103.30	103.30	....	98.21	98.21	....

Differences are marked with the minus sign for actual elevations falling below the straight line for high water, and for elevations falling above the straight line for low water.

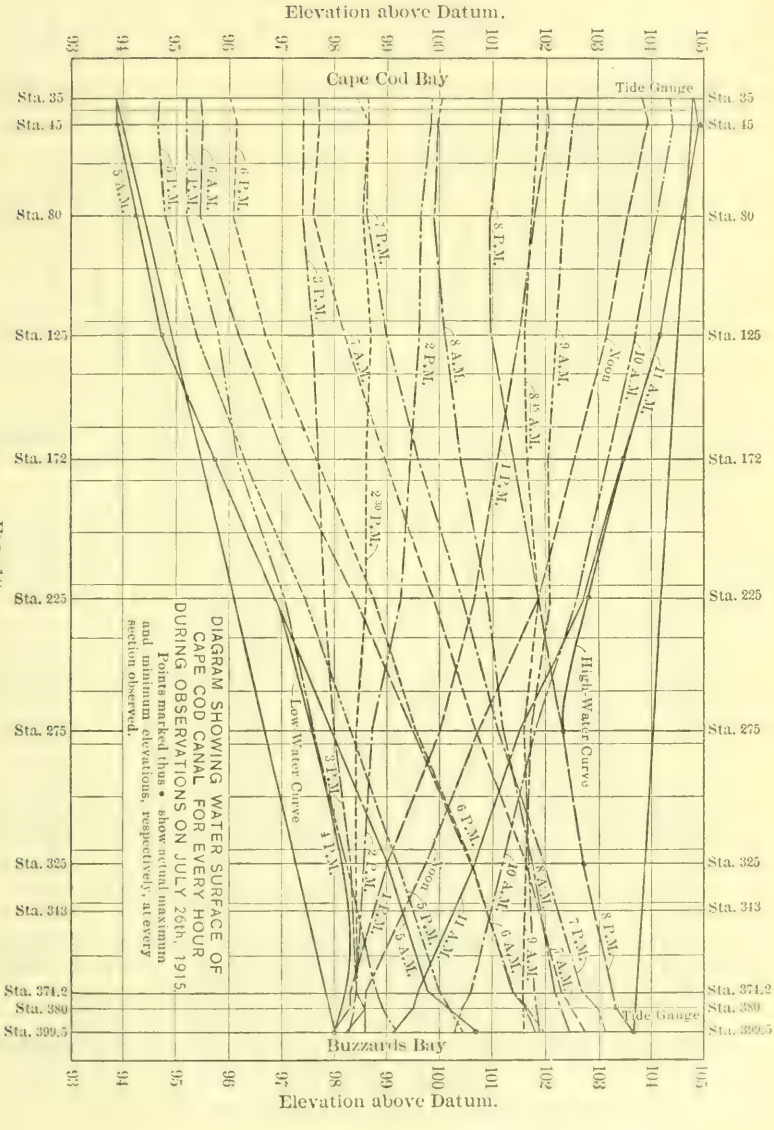

Elevation above Datum.
93 94 95 96 97 98 99 100 101 102 103 104 105
Cape Cod Bay
Tide Gauge
Sta. 35
Sta. 45
Sta. 80
Sta. 125
Sta. 172
Sta. 225
Sta. 275
Sta. 325
Sta. 343
Sta. 371.2
Sta. 380
Sta. 399.5
High-Water Curve
Low-Water Curve
Buzzards Bay
DIAGRAM SHOWING WATER SURFACE OF CAPE COD CANAL FOR EVERY HOUR DURING OBSERVATIONS ON JULY 26th, 1915.
Points marked thus • show actual maximum and minimum elevations, respectively, at every section observed.

FIG. 18.

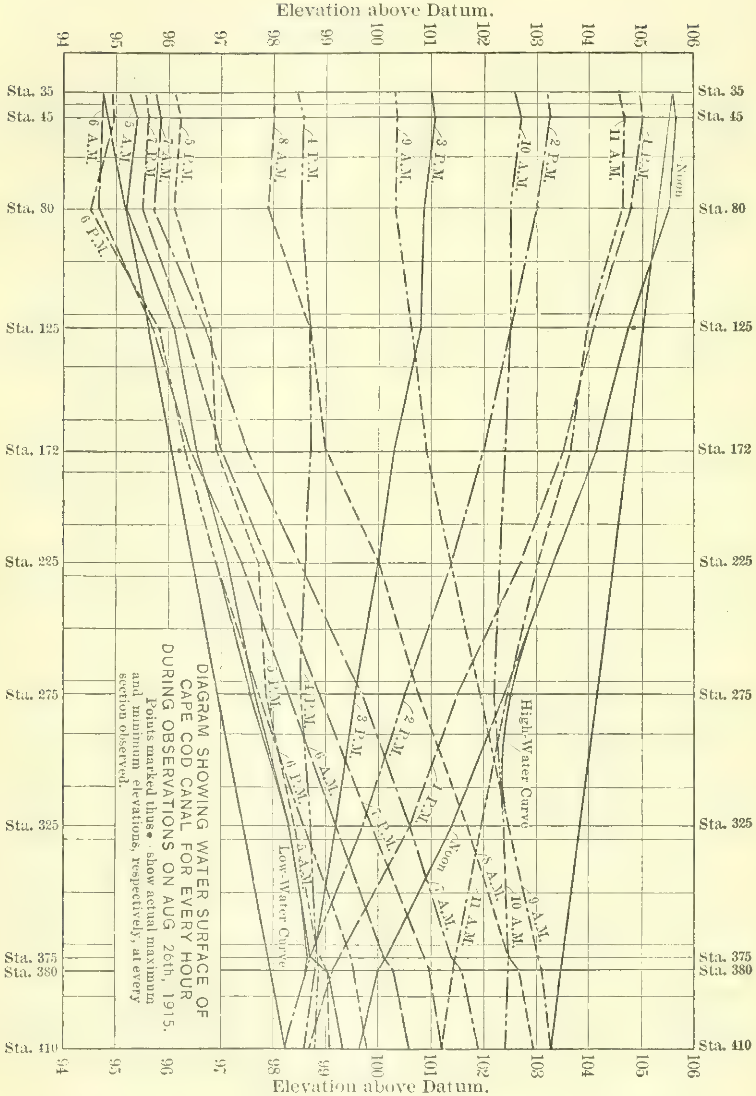

Elevation above Datum.
94 95 96 97 98 99 100 101 102 103 104 105 106
Sta. 35
Sta. 45
Sta. 80
Sta. 125
Sta. 172
Sta. 225
Sta. 275
Sta. 325
Sta. 375
Sta. 380
Sta. 410
6 A.M.
5 A.M.
7 A.M.
7 P.M.
5 P.M.
8 A.M.
4 P.M.
9 A.M.
3 P.M.
10 A.M.
2 P.M.
11 A.M.
1 P.M.
Noon
6 P.M.
High-Water Curve
Low-Water Curve
DIAGRAM SHOWING WATER SURFACE OF
CAPE COD CANAL FOR EVERY HOUR
DURING OBSERVATIONS ON AUG 26th, 1915.
Points marked thus • show actual maximum
and minimum elevations, respectively, at every
section observed.

FIG. 19.

A moment's consideration will explain this curious phenomenon: If, in an open channel, there is tidal action at one end only, that is, if the tidal range at the other end is zero, then elevations of high and low water at intermediate points will be on straight lines running from mean sea level at one end to high or low level at the other, the elevations being proportional to the distance. If, in the case of a comparatively short canal, where the instantaneous surface curves can be considered as substantially straight lines, there is tidal action at both ends, amplitude and phase being the same, then high, low, and intermediate elevations through the canal will be on horizontal straight lines; but, if the amplitude is the same and the phase is opposite, that is, if the tides are a full tide apart, then the elevations of high and low water through the canal would still be on straight lines, but broken at the midway point, the instantaneous profiles oscillating through this same point. In this case, no matter how great the rise and fall at the ends, there would be no tidal rise or fall at the center. When the amplitude and phase of the tides differ, the connecting lines become curved, as in the present case. This question of tidal elevations in the canal must not be confused with either the velocity or the instantaneous slope of the water.

In Fig. 18 the curves of high and low water are not symmetrical, which at first thought appears should be the case. The lack of symmetry is due to the incompleteness of the lower branch of the Buzzards Bay tide curve. If the tide fell below mean sea level as much as it rises above, the apex of the low-water curve would be advanced so as to lie beneath the apex of the high-water curve.

The elevation and location of the apices of the high- and low-water curves are functions of the phase difference and relative magnitudes of the terminal tides.

The line of elevations of low water is of great practical importance in construction. In order to give a uniform depth of water at low tide, it is not necessary to give a uniform slope to the bottom. The line of low-water elevations should be determined, which can easily be predicated from tidal records at the ends, and the excavation be made parallel thereto. An inspection of the figures in Table 12 indicates the variation from computed straight-line or uniform-slope elevations, and the corresponding saving in unnecessary excavation can be estimated.

The current velocities ascertained by the observations were the velocities on the surface at the center of the canal, and no other method of measuring velocities was practicable, in view of the required great frequency of the observations at so many stations, subject as they were to interruption by passing vessels. In order to harmonize the observations with flow formulas which depend on and give mean velocity, it was necessary to ascertain the relation existing between the center surface velocity and the mean velocity of the whole cross-section, as it actually existed in a channel of these dimensions and with frictional resistance produced by the material forming the sides and bottom.

The method adopted to determine the mean velocity and its ratio to the center surface velocity was that recommended by the U. S. Coast and Geodetic Survey Bureau. A point in the canal, Station 225, was selected where, for a considerable distance in both directions, the alignment was a tangent and the cross-sections of the canal were substantially uniform. The various threads of flow, therefore, were straight, parallel, and practically undisturbed by local eddies.

A wire was stretched across the canal, clear of the water, and on it were fastened tags at intervals of 10 ft. A boat, with the measuring crew, was held at each tag by an anchor while velocity readings were made, beginning at the surface and then downward at intervals of 2 ft. Measurements were thus made on a spacing of 10 ft. horizontally and 2 ft. vertically from shore to shore and from surface to bottom.

The gauging was done with a Gurley-Price current meter, which had been accurately rated by the manufacturers immediately prior to use. The revolutions of the meter were counted for 30 sec., and the recording of the velocities in each full vertical took from 10 to 20 min. each. In addition to recording the readings of the meter, all attending circumstances were noted, such as the elevation of the water, direction and strength of the wind, passing of vessels, etc., and float velocity determinations were made as a check on the meter and the work, and especially to record the variations in the surface velocity during the measurements.

The gauging was begun on the center line on July 10th, 1915, at 9.35 A. M. and proceeded toward the north bank, the vertical, 90 north, being measured at 11.51 A. M., when the elevation of the water surface had fallen from 101.7 to 100.5. Beginning again at vertical,

10 south, at 11.56 A. M., vertical 90 south was finished at 1.43 P. M., when the water surface stood at Elevation 99.6. During the latter half of the work the current had slackened considerably, so that the readings in the south half of the canal section were much lower than at corresponding points in the north half. In order to correct this discrepancy, readings in the south half were repeated 4 days later when local conditions of tide, current, and wind were comparably similar to those prevailing on the first occasion.

In spite of this repetition, which eliminated excessive variation in current condition, there naturally were minor variations, as developed over a period of more than 2 hours. These were compensated by reducing all observations to maximum center velocity by multiplying current-meter readings in each vertical by the ratio of maximum float velocity to the float velocity observed during the reading at such vertical, the float observations being repeated when the meter readings on each vertical were made.

For example, the maximum center velocity occurred when the current-meter readings were taken on the vertical, 30 north, and amounted to 2.96 knots. When the readings were made on the vertical, 60 north, the center surface velocity was 2.55 knots, and therefore all readings in this vertical were multiplied by the ratio, $\frac{2.96}{2.55} = 1.16$ in order to bring them to a parity with those in the vertical, 20, which were the maximum and standard.

The velocities, in knots, at points in the cross-section at horizontal intervals of 10 ft. and vertical intervals of 2 ft. and reduced to a common basis, are platted in Fig. 20. An average of these figures gives the mean velocity of the whole cross-section as 78% of the maximum surface velocity.*

In order that these observations should have all personal bias eliminated, and, so far as possible, receive official recognition, suggestions as to the method of obtaining the data accurately were invited from the U. S. Coast and Geodetic Survey Bureau, and that Bureau kindly delegated a representative, Homer P. Ritter, M. Am. Soc. C. E., to supervise the taking of some of the measurements, to see that they were properly made, and some measurements were repeated in order

* H. de B. Parsons, M. Am. Soc. C. E., in "Tidal Phenomena in New York Harbor," *Transactions*, Am. Soc. C. E., Vol. LXXVI, p. 2032, says: "The mean sectional velocity is about 0.75 times the velocity at the surface."

CAPE COD CANAL

CURRENT OBSERVATIONS, JULY 27TH, 1915.

W.L. El. 101.2 ±

Depth, in Feet, Below Water Surface	Sta. 90 South	Sta. 80 South	Sta. 70 South	Sta. 60 South	Sta. 50 South	Sta. 40 South	Sta. 30 South	Sta. 20 South	Sta. 10 South	C.L. Sta. 225	Sta. 10 North	Sta. 20 North	Sta. 30 North	Sta. 40 North	Sta. 50 North	Sta. 60 North	Sta. 70 North	Sta. 80 North	Sta. 90 North
0	0.95	1.70	1.95	2.10	2.06	2.58	2.76	2.89	2.92	2.46	2.95	2.74	2.70	2.57	2.43	1.97	1.39	1.18	0.94
2	0.95	1.51	1.81	2.10	2.32	2.67	2.46	2.86	2.80	2.80	2.86	2.96	2.94	2.66	2.18	2.16	1.67	1.20	1.01
4	1.00	1.59	1.76	2.32	2.36	2.46	2.86	2.86	2.95	2.96	3.19	2.60	2.94	2.71	2.33	2.31	1.56	1.67	1.07
6	1.10	1.59	1.67	2.13	2.49	2.63	2.76	2.86	2.98	2.90	3.04	3.11	3.04	2.85	2.46	2.50	1.89		0.59
8	0.90	1.45	1.90	2.13	2.53	2.63	2.93	2.93	3.05	3.03	2.99	3.01	2.66	2.71	2.43	2.50	2.09	1.67	1.72
10	0.73	1.20	1.49	1.88	2.28	2.89	3.21	3.16	3.10	3.10	3.04	3.06	3.04	2.85	2.21	2.11	2.19	1.67	1.72
12	0.79	0.91	1.76	1.75	2.49	2.94	2.76	3.08	3.05	3.10	3.23	3.16	3.04	2.90	2.78	2.31	1.77	1.27	1.60
14		0.12	1.72	1.78	2.41	2.84	2.84	2.84	3.02	3.03	3.01	3.01	2.89	3.00	2.73	2.44	1.33	1.72	1.12
16		0.48	1.45	1.70	2.25	2.86	2.73	2.84	2.82	2.98	2.99	3.20	2.80	3.06	2.78	2.05	1.41	0.96	
18			1.20	1.57	2.25	2.58	2.76	2.89	2.80	3.09	2.81	2.83	2.85	3.00	2.64	1.90	1.27	0.46	
20			0.51	1.40	2.10	2.58	2.98	2.89	2.92	3.03	3.04	2.51	2.85	2.76	2.84	2.22	1.67		
22				1.08	2.13	2.41	2.93	2.84	2.82	3.11	2.99	2.60	2.66	2.61	2.38	2.36	1.56		
24				0.87	1.66	2.41	2.76	2.84	2.90	2.78	2.49	2.92	2.85	2.71	2.60	2.31	1.77		
26					0.69	1.78	2.20	2.53	2.70	2.73	2.72	2.83	2.61	2.61	2.33	1.96	1.60		
28						0.61	2.13	2.24	2.81	2.86	2.77	2.64	2.28	2.15	2.06	1.96			
30							1.40	2.28	2.78	2.78	2.24	2.38	2.38	2.27	1.92	1.90			
32							0.56	1.49	1.79	2.36	2.21	2.10	2.28	2.06	1.57				
Mean velocities in verticals, in knots.	0.92	1.21	1.57	1.76	2.14	2.46	2.53	2.72	2.84	2.92	2.86	2.80	2.75	2.67	2.39	2.18	1.66	1.31	1.22

Figures to the right of verticals denote velocities, in knots. These velocities have been computed from the results of measurements taken on July 10th and 14th, 1915, and reduced to a common time point.

Total of all velocities — 620.18 knots

Number of observations — 269

Mean velocity — 2.31 knots

Mean velocity, in terms of maximum surface velocity — .787

FIG. 20.

to check the accuracy of those previously recorded. The Engineer of the Harbor and Land Commission of Massachusetts did likewise.

An inspection of Fig. 20 shows that the maximum velocity, as in other streams, is not at the surface, but at a considerable distance below it, and that the velocity increases from that at the surface to the maximum and then decreases with further increase in depth. The measured velocities in the seven central verticals, 30 south to 30 north, both inclusive, in which the velocities are the maximum, and which are the least affected by side resistance and eddies, have been averaged and platted in the solid line in Fig. 21.

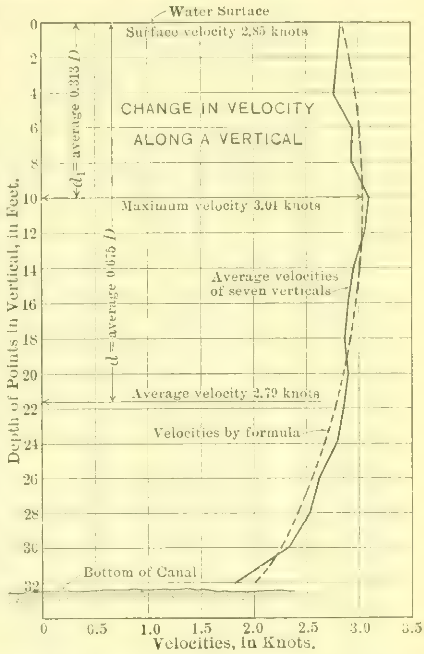


FIG. 21.

An analytical expression of the change in vertical velocity in a stream is given by the parabolic formula proposed by Capt. Humphreys and Lieut. Abbot, Topographical Engineers, U. S. A., in their memorable classic "The Hydraulics of the Mississippi River" in 1861:

$$V = V_{d_1} - (b\,v)^{\frac{1}{2}} \left(\frac{d - d_1}{D}\right)^2$$

where V = velocity at a point in the vertical the depth of which below the surface is denoted by d;

V_{d_1} = maximum velocity in the vertical;

v = mean velocity of the whole cross-section;

D = depth of vertical;

d_1 = depth of the point of maximum velocity in the vertical below the surface;

d = depth of any point in the vertical below the surface;

b = $\dfrac{\text{constant}}{\sqrt{D + 1.5}}$, the value of which for the section of the canal under consideration was found to be 2.93.

The curve given by this formula is shown by the dotted line in Fig. 22, its point of origin at the surface being taken coincident with the measured surface velocity. The measured vertical velocity line is, as was to be expected, a broken and not a regular curve, but it corresponds with astonishing closeness with the theoretical parabola, so closely that the latter is a fair average of the former, as it should be. The depth of maximum velocity is 0.313 D. The depth to the point on the parabola where the velocity is the same as the mean velocity is 0.675 D. In round numbers, the maximum velocity occurs at one-third, and the mean velocity at two-thirds, of the depth. The average maximum velocity is 1.066 times the maximum surface velocity, and therefore the mean velocity, being 0.78 of the maximum surface velocity, is 0.73 of the maximum velocity.

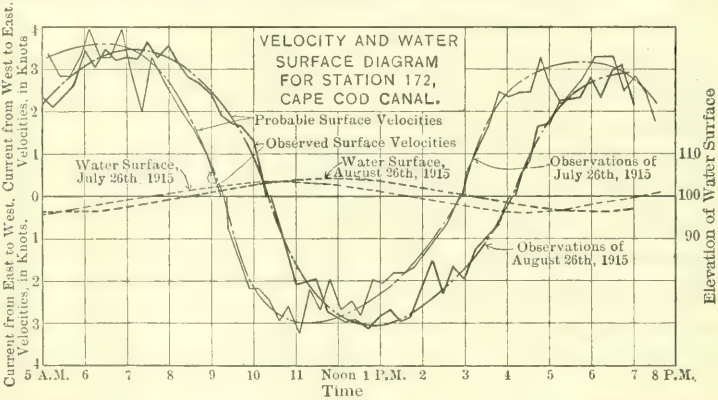


FIG. 22.

The plotting of the velocity observations gave curves which were not smooth but broken. These breaks in alignment were due to the inevitable variations in contiguous observations and to the fact that water in a large channel does not flow with absolute uniformity but in pulsations developed from any causes, such as the roughness of the bottom and sides, producing eddies; the local action of wind, and the passing of boats setting up waves that are felt at considerable distances. These irregularities were disregarded, and a smooth curve in each case was plotted, which was the average of the observations. As a matter of interest, Fig. 22 shows the actual observations as made at Section 172 (a fair example) and the smooth curve which was taken as the basis for computation.

Figs. 23 to 27 show the smooth curves at each of the observation stations, the right-hand scale referring to center surface velocities, the left-hand scale to the velocities reduced to mean velocity for the whole section by multiplying the observed surface velocities by 0.78, the ascertained coefficient. The abscissas of these curves represent

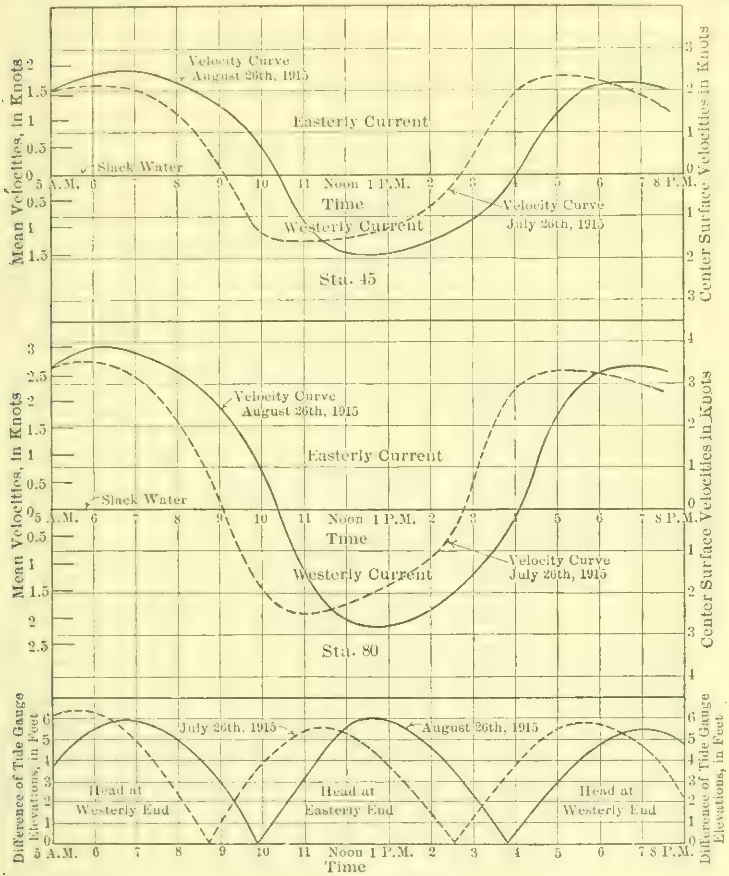


FIG. 23.

the time, and the velocities, in knots* per hour, are plotted as ordinates, above the axis for easterly and below for westerly current. Where the direction of the current is referred to as easterly or westerly, it is to be understood as being toward the east or west; that is, from Buzzards Bay to Cape Cod Bay or the reverse, respectively. The

* 1 knot = 1 nautical mile = 6 080 ft., 1 knot per hour = 1.69 ft. per sec.

observations on both occasions (July 26th and August 26th) are shown with evident closeness in results.

To show the relation existing between velocity and "head", curves are plotted at the bottom of each figure showing the measured differences in water elevations at the two ends of the canal, also plotted as ordinates to the time abscissas.

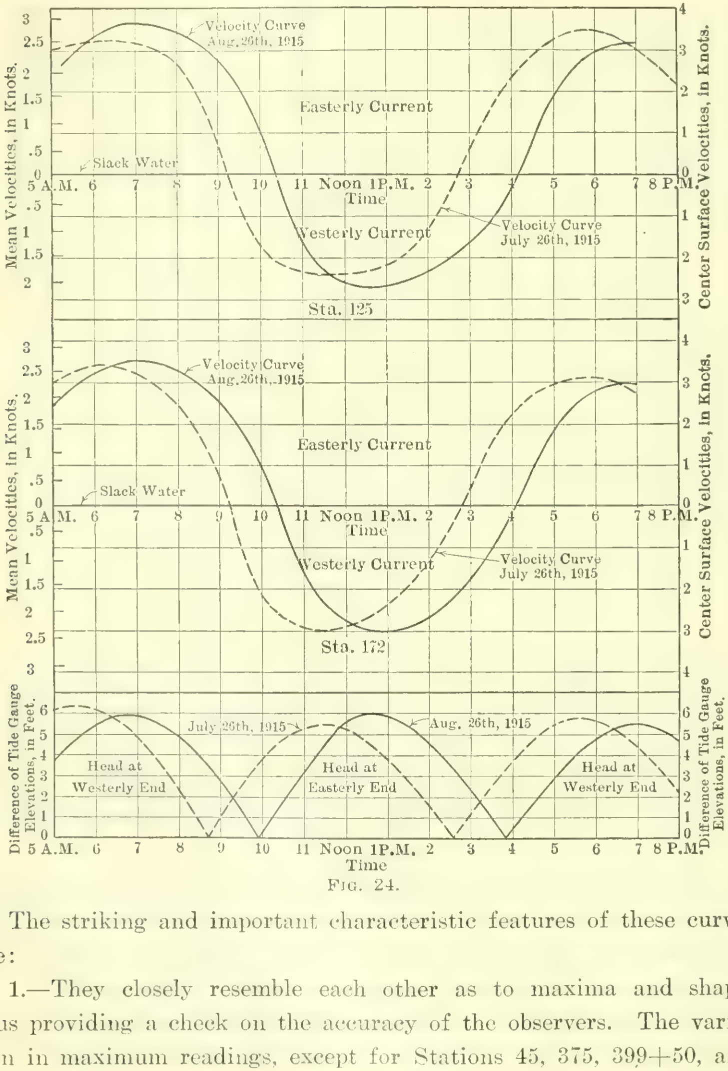


Fig. 24.

The striking and important characteristic features of these curves are:

1.—They closely resemble each other as to maxima and shape, thus providing a check on the accuracy of the observers. The variation in maximum readings, except for Stations 45, 375, 399+50, and

410, where the canal cross-section is greatly increased, and at Station 80 on the easterly and Station 375 on the westerly current, to be referred to later, are readily accounted for by slight variations in local conditions.

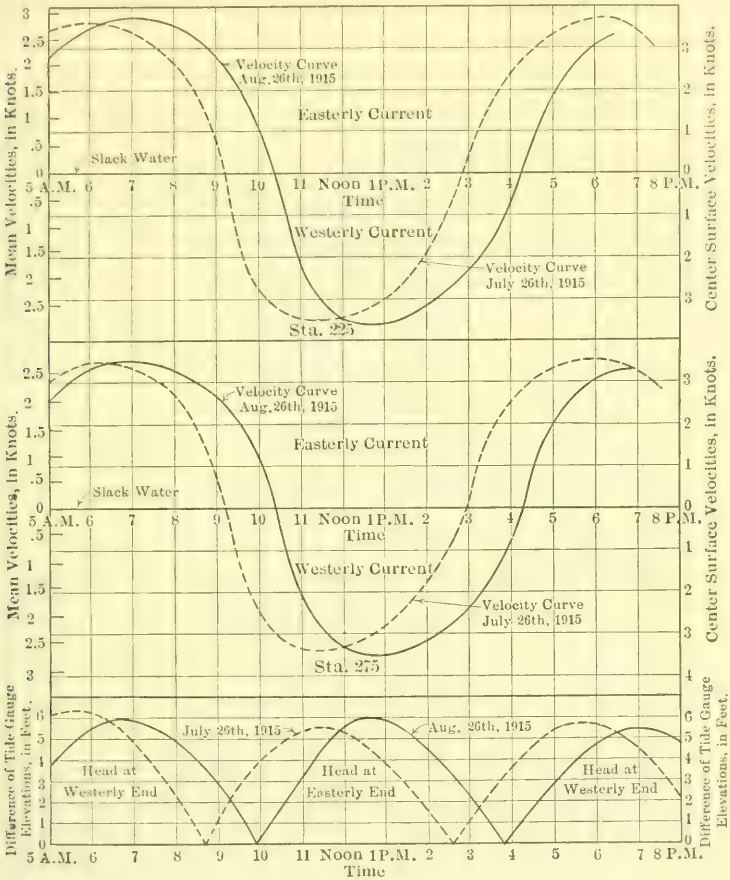


FIG. 25.

2.—The form of the curve approximates that of a sine curve, that is, the rate of change of the absolute value of the velocities is zero at the maximum in both directions of flow, and increases gradually toward zero velocities. The change of sign takes place very rapidly at the reversal of the current. At maxima velocities the character of flow approximates that of uniform motion.

3.—Maximum and zero velocities occur at very nearly the same instants throughout the whole length of the canal. These last two features indicate that, for the special case of the Cape Cod Canal, good approximate values for maxima velocities can be derived by applying formulas pertaining to uniform—or at least permanent—flow in channels.

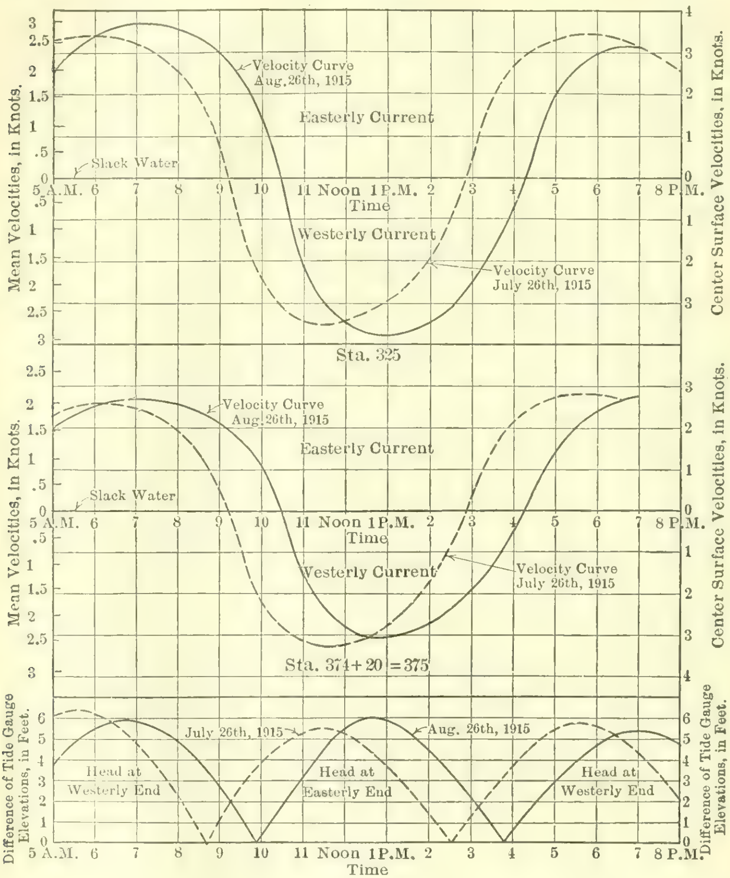


Fig. 26.

4.—The descending branches of the velocity curves have a somewhat greater inclination than that of the ascending branches.

5.—The duration of the easterly current is sensibly longer than that of the westerly current.

The diagram of elevation differences shows that though, for some local cause, the maximum head was considerably lower for westerly than for easterly currents during the observations on July 26th, on August 26th the maxima heads in opposite directions were almost equal. Therefore, for the theoretical analysis of the problem, the data furnished by the observations on the latter date have been selected.

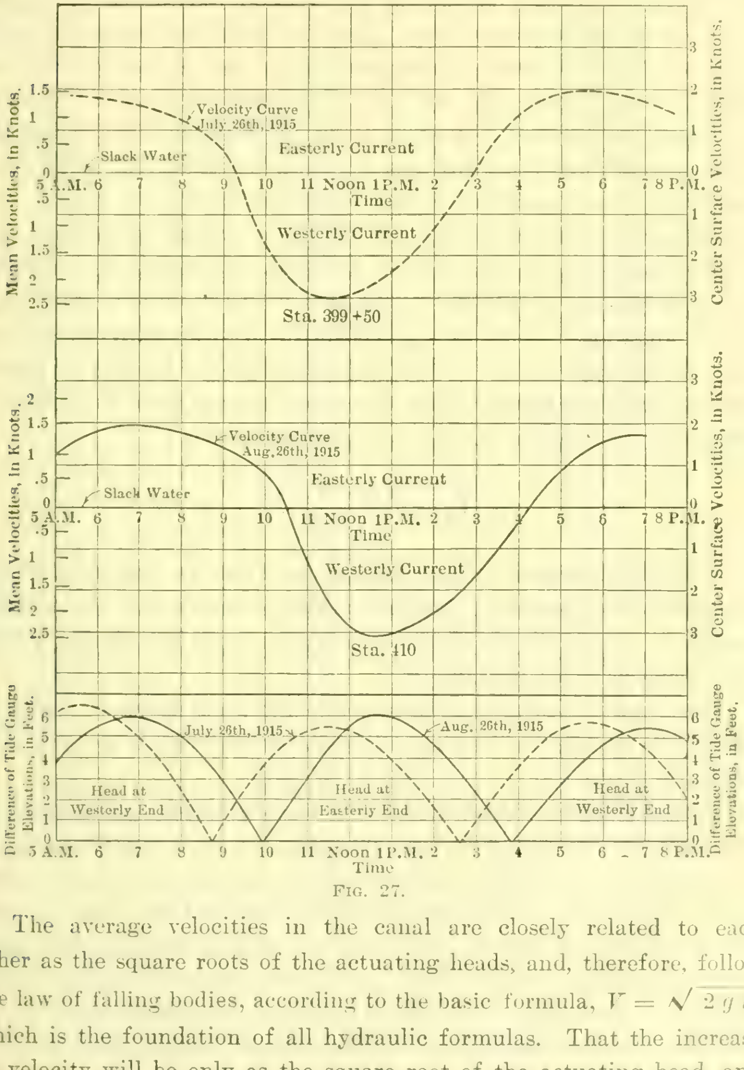


FIG. 27.

The average velocities in the canal are closely related to each other as the square roots of the actuating heads, and, therefore, follow the law of falling bodies, according to the basic formula, $V = \sqrt{2\,g\,h}$, which is the foundation of all hydraulic formulas. That the increase in velocity will be only as the square root of the actuating head, and

not as the full head, was a fact overlooked by many in considering the possibility of opening a canal at sea level without locks. Although admitting the possibility of success for the Cape Cod Canal at heads of 5 ft., some people had serious doubts as to what would happen at times of great storms piling up the water at one end and depressing it at the other, their fears extending even to the complete destruction of the canal by excessive erosion of the banks. Experience has shown that, even on extraordinary tides, when the head has reached a maximum of 9 ft., the current, though swift, is neither destructive nor prohibitory to navigation. At the time of the occurrence of the observed maximum head during the great storm of January, 1915, when the water surface at the east end was 9.5 ft. higher than that at the west end, the central surface velocity was measured as 4 knots, corresponding to a mean velocity of 3.12 knots, which is in line with the increase as the square root of the head. Therefore, to determine the mean velocity in the Cape Cod Canal at any head, it is only necessary to know the velocity at a given head and to multiply that by the ratio of the square roots of the head at which the velocity is sought and that which is given. Thus, the mean velocity at 5 ft. of head is 2.36 knots. The mean velocity at any other head is

$$V_h = \frac{2.36 \sqrt{h}}{\sqrt{5}}.$$

It has been pointed out, as will be seen from an inspection of the diagrams, that the head remains nearly constant for considerable lengths of time, the actual head on a typical mean tide during a tidal cycle being as given in Table 13.

The mean velocities corresponding to the heads given in Table 13, taking the average of the observations on both days, are given in Table 14.

Another feature of interest is the following: Although the velocity diagrams for Stations 172, 225, 275, and 325 show practically equal maxima in both directions, as is to be expected from equal tidal differences, the readings at the terminal stations (80 and 399 + 50) on July 26th or at Station 410 on August 26th, show considerable discrepancy between maxima. Station 80 is at the east end of the canal proper, close to the point where it is doubled in bottom width; Stations 399 + 50 and 410, although possessing a greater area of cross-section than the

TABLE 13.—Tidal Differences on August 26th, 1916.

Time, in hours and minutes.	Head, in feet.		Mean velocity in canal, in knots.	
5.00 A. M.	Head at westerly end	3.75	Easterly current	2.03
5.15		4.35		2.21
5.30		4.75		2.35
5.45		5.17		2.48
6.00		5.50		2.57
6.15		5.83		2.64
6.30		5.87		2.70
6.45		5.89		2.72
7.00		5.72		2.72
7.15		5.58		2.71
7.30		5.42		2.68
7.45		5.10		2.63
8.00		4.70		2.55
8.15		4.42		2.45
8.30		4.08		2.33
8.45		3.46		2.19
9.00		2.83		2.02
9.15		2.15		1.83
9.30		1.42		1.58
9.45		0.36		1.27
10.00	Head at easterly end	0.12		0.89
10.15		0.92		0.41
10.30		1.67	Westerly current	0.24
10.45		2.42		0.87
11.00		3.12		1.39
11.15		3.96		1.76
11.30		4.58		2.02
11.45		5.17		2.21
Noon		5.58		2.34
12.15 P. M.		5.83		2.43
12.30		5.96		2.49
12.45		5.92		2.51
1.00		5.83		2.51
1.15		5.66		2.47
1.30		5.42		2.42
1.45		5.08		2.34
2.00		4.58		2.23
2.15		4.12		2.11
2.30		3.50		1.95
2.45		2.83		1.76
3.00		2.21		1.56
3.15		1.66		1.32
3.30		1.00		1.05
3.45		0.25		0.76
4.00	Head at westerly end	0.42		0.41
4.15		1.00	Easterly current	0.02
4.30		1.62		0.55
4.45		2.33		1.08
5.00		2.92		1.47
5.15		3.42		1.74
5.30		3.92		1.97
5.45		4.30		2.14
6.00		4.58		2.27
6.15		4.95		2.36
6.30		5.25		2.43
6.45		5.42		
7.00		5.42		
7.15		5.42		
7.30		5.30		
7.45		5.00		
8.00		4.75		

normal canal section, are at the point where the canal debouches into Buzzards Bay. When the current was flowing east, the maximum center surface velocity on August 26th at Station 80 was 3.8 knots, as compared with 2.8 knots when the current was flowing west; whereas,

TABLE 14.

Head, in feet.	Observed mean velocity in canal, in knots.	Mean velocity by formula, $2.36 \times \frac{\sqrt{h}}{\sqrt{5}}$ knots.
3	1.69	1.83
3½	1.93	1.98
4	2.03	2.12
4½	2.20	2.24
5	2.36	2.36
5½	2.50	2.48
6	2.62	2.60

at Station 225, on the same date, the maximum center surface velocity was 3.7 knots in each direction. At the west end of the canal at Station 410 the maximum center velocity on the east-bound current was less than 2 knots and slightly more than 3 knots on the west-bound current.

That is to say, in both cases when the current was flowing from a narrow to a broader cross-section, the center surface velocity was increased; and it was decreased when the flow was in the opposite direction, or from broad to narrower. As the volume of water passing Station 80 is the same as in any other normal canal cross-section, though the center velocity is quite different, it must be that the same relation between center surface velocity and mean velocity does not exist.

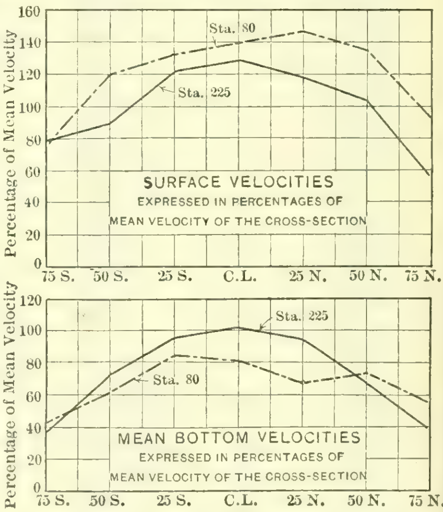


FIG. 28.

To prove this assumption, current-meter measurements were made at Stations 75 and 85, and the average of these showed that the mean velocity of the cross-section at Station 80 is only about 68%, instead of 78%, of the center surface velocity when the current is flowing in an easterly direction. In Fig. 28 the velocities on the surface and near the bottom of the canal are plotted, expressed in percentages of the mean velocity for the whole cross-section for Stations 80 and 225, for

comparison. This diagram shows that though the surface velocities at Station 80 are measurably greater than those at Station 225, the velocities near the bottom are in the inverse ratio. No measurements were made on the westerly flow, but undoubtedly the opposite conditions exist; that is, there is a higher proportionate bottom and side velocity to balance the known lower surface velocity, so that the mean cross-sectional velocity would have a higher ratio to the center surface velocity than the normal figure of 0.78, as that on the easterly flow is lower, the ratio being about 0.85 to 0.88. As this phenomenon is repeated at both ends of the canal, it is evidently not a matter of chance, and an explanation is offered that, when the current is from a narrow to a broad section, there is a contraction similar in character to that which occurs when a jet issues from an orifice; but when the current is from broad to narrow the diverted threads of flow at the sides, which find no place in the narrow section, if continued in straight lines, are pushed toward the center, and increase the velocity of flow at the sides, thus automatically reducing the velocity of the surface center flow, as the volume of water passing the section in a given interval of time is the same; and if the velocity at any part of the cross-section is increased, a compensative reduction must take place at some other point.

In a Canal, therefore, the transition to increased cross-section should be made very gradually. In the canal in question a length of 500 ft. was given to a gradual widening, but this is seen to be insufficient to eliminate all variations in flow.

A phenomenon of much interest, as shown by these diagrams, is that the time of neither maximum current nor zero current coincides with the time of maximum tidal difference or of simultaneous equal end elevation.

Averaging the times of maximum and zero currents at Stations 80 to 375, both inclusive, so as to eliminate local variations in observations, the lag in time for the foregoing current conditions is:

Maximum easterly current behind Buzzards Bay high. .0.31 hour.
Maximum westerly current behind Cape Cod Bay high. .0.21 "
Zero velocity current behind equal elevation, Buzzards Bay tide falling ....0.51 "
Zero velocity current behind equal elevation, Cape Cod Bay tide falling ....0.44 "

By "Buzzards Bay tide falling" or "Cape Cod Bay tide falling" is meant that the tide at the end named is ebbing, and what had been superelevation at that end immediately prior to the moment of equal end elevation is about to be reversed.

It will be noted that the lag of zero velocity in both cases is greater than the lag of maximum velocity, and that the lag of the maximum or minimum (= 0) state of the current when governed by Cape Cod tidal influence is greater than when governed by Buzzards Bay tidal influence. An examination of the current diagrams shows in all cases a steeper inclination to the descending than to the ascending part of the curve, which accounts for the difference in lag.

A singular outcome of this phenomenon is that, with equal end elevations following Buzzards Bay falling tide, there is a current with an average mean velocity of more than 1 knot per hour, although at that instant there is no head to produce it. When the current velocity is zero (or slack water), there is a difference in head in Cape Cod Bay over Buzzards Bay of 1.5 ft. Between those limits of time (30 min.) the water in the canal is actually running up hill, from a maximum current of 1.1 knots when the slope is level to a current of zero velocity when the adverse head is 1.5 ft. On the other tide, when the end elevations are equal, the mean current velocity is 0.74 knot, and when the velocity is zero the adverse head, opposite to what has been producing flow immediately previous, is 1.0 ft.

The explanation of these phenomena of lag lies in the dynamic properties of moving liquids. A time interval is required to develop full momentum imparted by an extraneous force, in this case the full momentum not being reached until after its creating force has passed the apex of its energy. In like manner, momentum when once set up continues unaided until absorbed by friction or checked by a new and opposing force. The case of an inflowing tide in a narrow estuary being stopped by a dam or lock and producing at that point a higher elevation to high tide and a lower one to low tide than the normal is well known. In the Cape Cod Canal there is no such abrupt stop, but there is seen a very beautiful illustration of the balancing and oscillating action of two waves in their alternate development and arresting of motion. The rising tide at one end, when its elevation exceeds the tide at the other, produces a force in opposed head to overcome the momentum imparted by the previous "head" at the other end, and

then it in turn establishes a return flow. It will be noted that, when the velocity at 0 ft. differential of elevation is 1.1 knots, an increasing head to a maximum of 1.5 ft. is necessary to check it, and, when it has the opposite direction of velocity of 0.74 knot, a head of 1.0 ft. suffices. On account of this lag, the diagrams must be read with care to determine the velocity corresponding to any given head. A direct projection from the "head" to the "velocity" curve will not give the accurate rate; allowance must be made for lag. The harmonic analysis of this and other features of the problem is presented in the mathematical consideration.

A further peculiarity of the motion of the water in the Cape Cod Canal is revealed by inspecting Fig. 19, which represents the instantaneous surface curves of the canal for August 26th, 1915. It will be noted that at Station 172 the inclination of the surface had exactly the same value at noon (westerly current) as at 6.00 P. M. (easterly current), but the elevations of the water were 104.13 and 96.30, respectively, the hydraulic radius being 22.25 ft. at noon and 17.95 ft. at 6.00 P. M. From the principles of hydraulics, therefore, the velocity of the flow at noon should have been about 16% greater than at 6.00 P. M. $\left(\frac{v_1}{v_2} = \frac{C_1 \sqrt{R_1 S}}{C_2 \sqrt{R_2 S}} = 1.16\right)$. However, referring to the corresponding velocity diagram, it will be found that the velocities were almost exactly the same at these two time points. A similar result is obtained for Station 225, comparing observations at 7.00 A. M. and 1.00 P. M. This fact indicates that a change of the hydraulic radius, resulting solely from the tidal variations of the depth, has only a slight influence on the velocities, and that the flow in tidal streams similar to the Cape Cod Canal obeys different laws than those governing the uniform motion of water in canals.

Another unique feature, the discussion of which is made possible by the measurements of water elevations at the Cape Cod Canal, is the very complicated method of the wave propagation therein. In a canal subjected to tidal influences at one end only, the propagation of the wave is at a nearly constant rate, and its velocity can be expressed approximately by $w = \sqrt{g\,H}$. (This formula will be discussed later in the mathematical part of the paper.) Measurements in the Suez Canal

showed a very close agreement between the computed and observed values of the velocity of wave propagation.

At the Cape Cod Canal the conditions are entirely different, the value of the velocity of wave propagation being a function of the time interval between the high-water points of the two ends and also of the relative magnitude of the amplitudes of the two tides. It is evident that high water along the canal at the consecutive stations will be reached in the time interval between the high waters at Buzzards Bay and Cape Cod Bay. On August 26th this time interval was about 3 hours 20 min., high water at Station 410 occurring at 8.45 A. M. and at Station 35 at 12.05 P. M. The corresponding average velocity is 3.12 ft. per sec., or only about one-tenth of that furnished by the formula.

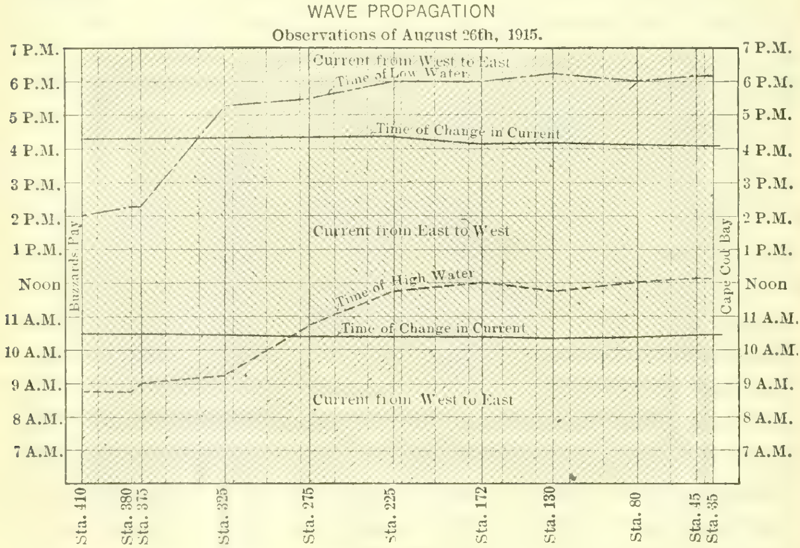


FIG. 29.

An inspection of Figs. 29 and 30, which show the time occurrence of high and low water and also of slack water along the canal, plotted as ordinates to the station abscissas, reveals the fact that the true propagation of the compound wave which results from the combination of the two tidal motions is very different from this average value. The propagation is very swift and sometimes almost instantaneous for the reaches near the ends of the canal, and it becomes very slow for about two-fifths of the length at high water and for about one-sixth at low

water. Figs. 29 and 30 show the overwhelming influence of the Cape Cod wave over the Buzzards Bay wave, the length of the corresponding nearly horizontal lines indicating time of high and low water along the canal being nearly proportional to the difference in amplitudes at these points. An interesting contrast to the propagation curve of high and low waters is the almost exactly simultaneous occurrence of slack water on the entire length of the canal, which is also shown on the diagrams.

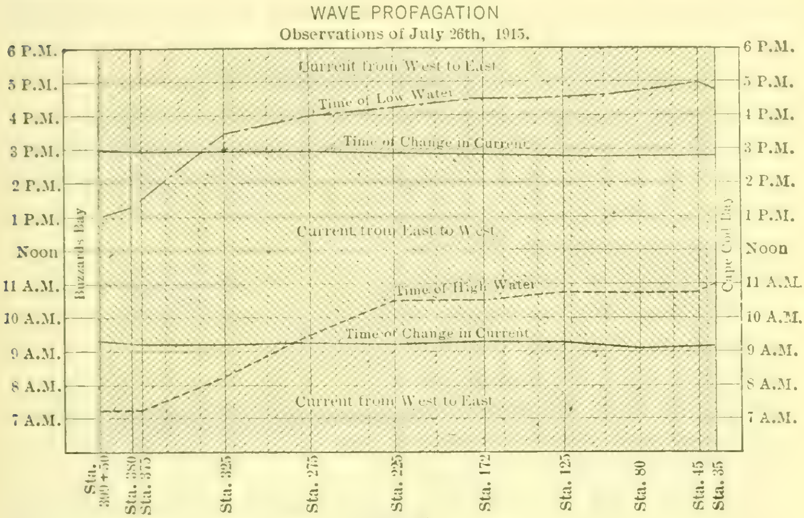


FIG. 30.

Although it does not seem possible to find an analytical expression for the velocity of the wave propagation in the Cape Cod Canal, the problem can be solved with the aid of the harmonic analysis of the motion, by calculating the water elevation curve for every station (as will be shown later) and picking out the high, low, and slack-water times of the same and plotting these times as ordinates to the station abscissas.

THE SOLUTION OF THE PROBLEM BY HARMONIC ANALYSIS.

It is a somewhat curious fact that textbooks on hydraulics in use in engineering practice do not give any information whatsoever on the analytical side of tidal phenomena, and that even standard books on hydrodynamics deal with the practical problem of tidal currents in a very incomplete way. For the study of tidal motion in canals, one must search the libraries for very rare scientific publications,

and, therefore, it is hoped that a complete discussion of the question will be welcomed by the members of the Society and the readers of the *Transactions* in general.

The classical solution of the problem of tidal currents in canals was derived by Sir George Biddell Airy, of Cambridge, later Astronomer Royal, Greenwich Observatory, in his dissertation on "Tides and Waves."* All subsequent treatment of the subject has been based on Airy's work, and in the following pages the theory of tides in canals, as derived by him and interpreted by Professor Maurice Lévy of the *Collège de France* in Paris, will be given with such modifications and additions as are deemed justified, with reference to the data obtained by careful measurements at the Cape Cod Canal.

1.—General Assumptions.—Assume that the canal is narrow enough to permit all movements to be regarded as parallel to the longitudinal axis of the canal, so that it is sufficient to study the conditions along a vertical plane through this axis. In Fig. 31 let F be the intersection of this plane with the bottom of the canal, which is supposed to be horizontal or very slightly inclined. Let S_0 be the intersection of the plane with the surface of the canal, when the water is in equilibrium or in a state of permanent motion, due to gravity and friction only, and let S be the intersection of the plane with the mobile surface, in case the motion of the water is not permanent, due to the action of the stars or any other cause.

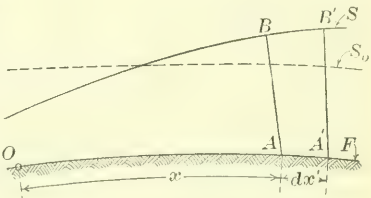


FIG. 31.

Also assume that, at the same line, A-B, normal to the bottom, the longitudinal displacements will be practically the same for every particle. This has been proved by Airy, who in Section 180 says:

"When the length of the wave is great in comparison with the depth of the water (as in the case of tide waves), the horizontal motion is sensibly the same from the surface to the bottom * * *."

It is sufficient, therefore, to study the motion of the point, B, at the surface.

Assume that the tide generated within the canal is so small as to be negligible, and therefore the non-permanent motion, which it is proposed to investigate, is due to other direct causes.

* "Encyclopædia Metropolitana," Vol. V, 1845.

Finally, assume that the movements of the line, S, are small in relation to the depth of the canal.

2.—Differential Equations of the Varying Motion of the Water in Canals.—Hydrostatics teaches that for a fluid at rest the free surface is a surface normal to the resultant of the forces at each point of application, B. According to d'Alembert's theorem, the same holds good for the case of motion, but the force of inertia of the point considered should be added to the attacking forces. The algebraic sum of the projections of these forces on the tangent to the surface is consequently zero.

Let j_t be the projection of the acceleration of the point, B, and m its mass; then $-\, m\, j_t$ is the projection of its force of inertia. The component of the gravity is $-\, m\, g \sin. I_s$, I_s, denoting the inclination of the tangent, BB' of the free surface, directed in the sense of increasing x' and being taken positive above the horizon, x' representing the distance of the section, AB, from a fixed point, O.

Then, disregarding friction, the equation sought will read

$$-\, m j_t - m\, g \sin. I_s = 0,$$

or

$$j_t = -\, g \sin. I_s \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (a)$$

being an expression of the mean acceleration along the section, AB.

Taking friction into consideration, the mean acceleration in the section determined by the friction must be added to the right-hand side of Equation (a). Now, the sum of the friction between the filaments of water is zero, by virtue of the principle of action and reaction, and therefore only the friction of the water at the wetted perimeter should be considered. Let F be the friction per unit of wetted surface and X the wetted perimeter of the section, AB, and let the section, $A'B'$, be infinitely near to AB, at a distance, $x' + d\, x'$, from O. The wetted surface of the canal between the two sections equals $X\, d\, x'$, and the corresponding friction, $F\, X\, d\, x'$. The mass of liquid, AB-$A'B'$ is $\frac{\eta}{g}\, \Omega\, d\, x'$, denoting the area of the section, $A\, B$, by Ω, and the weight of the liquid per unit volume by η.

The mean acceleration due to the force, $F\, X\, dx'$, therefore, reads

$$\frac{F\, X\, d\, x'}{\frac{\eta}{g}\, \Omega\, d\, x'} = \frac{g\, F\, X}{\eta\, \Omega}$$

and, being directed opposite to the motion, Equation (a) becomes

$$j_t = -g \sin. I_s - \frac{g F X}{\eta \Omega} \dots\dots\dots\dots\dots\dots (b)$$

Let y_f be the ordinate of the point, A, of the bottom of the canal, measured from an arbitrary datum line; this ordinate is a function of the one variable, x'; and let z be the depth of the canal at A at the instant, t, that is, z is a function of the two independent variables, x' and t. Then

$$\sin. I_s = \frac{\delta (y_f + z)}{\delta x'} = \frac{\delta y_f}{\delta x'} + \frac{\delta z}{\delta x'} = \sin. I + \frac{\delta z}{\delta x'} = I + \frac{\delta z}{\delta x'}$$

denoting the slope of the bottom by I, figured positive above the horizon, so that I is positive or negative according to whether, by ascending the bottom slope, we go in the direction or against the positive, x'.

Assuming that the friction, F, per unit surface is proportional to the nth power of the velocity, v, so that

$$F = f_0 v^n \dots\dots\dots\dots\dots\dots\dots\dots (c)$$

f_0 being a coefficient varying with the consistency of the wetted perimeter and being equal to F for unit velocity, Equation (b) becomes

$$j_t = -g I - g \frac{\delta z}{\delta x'} - f v^n \dots\dots\dots\dots\dots\dots (d)$$

if, for the sake of simplicity, we put

$$f = \pm \frac{g f_0 X}{\eta \Omega} \dots\dots\dots\dots\dots\dots\dots\dots (e)$$

In Equation (a), f is positive always, if n is an odd figure, also, if n is even and $v > 0$; in other words, if the motion is in the direction of positive x'. If $v < 0$ and n is even, the lower sign should be taken.

In order to transform Equation (d) into an equation of partial derivatives, let us characterize a section by the abscissa, x, and the depth of the water at that section by γ at a particular instant, which is taken as the origin of time; the abscissa, x', and the depth, z, of the section at the instant, t, will then be functions of the two independent variables, t and x. To follow the motion of the particles of a section designated by its initial abscissa, x, it is sufficient to let the

time vary only. The expression for velocity and acceleration will consequently be

$$v = \frac{\delta x'}{\delta t}, \text{ and}$$

$$j_t = \frac{\delta v}{\delta t} = \frac{\delta^2 x'}{\delta t^2} \dots\dots\dots\dots\dots\dots\dots\dots (f)$$

Substituting in Equation (d)

$$\frac{\delta^2 x'}{\delta t^2} + f\left(\frac{\delta x'}{\delta t}\right)^n + g\, I = -g\frac{\delta z}{\delta x'},$$

and, multiplying by $\frac{\delta x'}{\delta x}$, we get

$$\left[\frac{\delta^2 x'}{\delta t^2} + f\left(\frac{\delta x'}{\delta t}\right)^n + g\, I\right]\frac{\delta x'}{\delta x} = -g\frac{\delta z}{\delta x} \dots\dots\dots\dots (I)$$

which is one of the differential equations between the unknowns, x' and z.

The practical incompressibility of the water furnishes the other equation.

Ω being the area of the wetted cross-section at AB, let Ω_0 be the area of this same section at the initial instant. The volume of water included between this and an infinitely near section, having an abscissa, $x + d\,x$, is $\Omega_0\, d\,x$. At the instant, t, the abscissas of the two sections are x' and $x' + \frac{\delta x'}{\delta x} d\,x$, respectively. The volume included between them equals $\Omega\,\frac{\delta x'}{\delta x} d\,x$, which must be equal to $\Omega_0\, d\,x$, or

$$\Omega\frac{\delta x'}{\delta x} = \Omega_0 \dots\dots\dots\dots\dots\dots\dots\dots (II)$$

which is termed the equation of continuity.

3.—Application of the General Differential Equations to a Canal of Uniform Rectangular Cross-Section.—Let v_0 be the absolute value of the velocity in the canal for uniform motion, determined by the bottom slope and friction only. Then, for this condition,

$$x' = x - v_0 t, \text{ and } z = \gamma \dots\dots\dots\dots\dots\dots (f_a)^*$$

For the non-permanent oscillating motion,

$$x' = x - v_0 t + \xi, \text{ and } z = \gamma + h \dots\dots\dots\dots (g)$$

where ξ and h represent the deviations of x' and z from the conditions of uniform motion, and therefore are assumed to be comparatively small.

* The origin of the abscissas can always be selected in such a way, that v_0 shall be negative.

Assume, further, that for the range of velocities here considered the power of the velocity, to which the friction is proportional, is equal to unity; then Equation (I) can be written

$$\left(\frac{\delta^2 x'}{\delta t^2} + f \frac{\delta x'}{\delta t} + g I\right) \frac{\delta x'}{\delta x} = - g \frac{\delta z}{\delta x} \qquad \text{..........} (h)$$

which, for permanent uniform flow, reduces to

$$- f v_0 + g I = 0 \qquad \text{.....................} (i)$$

and substituting the value of $g I$ from Equation (i) in Equation (h), we get

$$\left[\frac{\delta^2 x'}{\delta t^2} + f \left(v_0 + \frac{\delta x'}{\delta t}\right)\right] \frac{\delta x'}{\delta x} = - g \frac{\delta z}{\delta x} \qquad \text{..........} (j)$$

Now, considering ξ and h as variables, as defined by Equation (g).

$$\frac{\delta^2 \xi}{\delta t^2} + f \frac{\delta \xi}{\delta t} = - g \frac{\frac{\delta h}{\delta x}}{1 + \frac{\delta \xi}{\delta x}} \qquad \text{...............} (k)$$

The cross-section being rectangular and uniform, $\Omega_0 = b_0 \gamma$, and $\Omega = b_0 z$, b_0, the width, being constant.

Therefore, from Equation (II),

$$\frac{z}{\gamma} = \frac{1}{\frac{\delta x'}{\delta x}} \qquad \text{........................} (l)$$

and again taking ξ and h as the variables, as defined by Equation (g), we get

$$z = \gamma + h = \frac{\gamma}{1 + \frac{\delta \xi}{\delta x'}} \qquad \text{.................} (m)$$

Now Equation (k), with respect to this last equation, can be written

$$\frac{\delta^2 \xi}{\delta t^2} + f \frac{\delta \xi}{\delta t} - g \gamma \frac{\frac{\delta^2 \xi}{\delta x^2}}{\left(1 + \frac{\delta \xi}{\delta x}\right)^3} = 0 \qquad \text{.............} (n)$$

and, neglecting the powers of $\frac{\delta \xi}{\delta x}$, as a first approximation, we finally have

$$\begin{cases} \dfrac{\delta^2 \xi}{\delta t^2} + f \dfrac{\delta \xi}{\delta t} - g \gamma \dfrac{\delta^2 \xi}{\delta x^2} = 0 \qquad \text{.............} (III)^* \\ h = - \gamma \dfrac{\delta \xi}{\delta x} \qquad \text{........................} (IV)^* \end{cases}$$

* See also Harris, "Manual of Tides," Part V, pp. 294 and 295.

It should be noted that these equations, although derived for a canal of rectangular cross-section, can be used, with the same degree of approximation, for uniform canals of any cross-section, supposing that by γ the "reduced depth" of the canal is understood, that is, the depth of a rectangular section having the same area and top width as the actual section. Substituting the value, $\Omega \doteq \Omega_0 + b_0\, h$, and the value of x' from Equation (g) in Equation (II), we get

$$\Omega_0 \left(1 + \frac{\delta\, \xi}{\delta\, x}\right) + b_0\, h = \Omega_0.$$

Let $\Omega_0 = b\, \gamma$, γ being the reduced depth, as explained, then $\gamma\, \frac{\delta\, \xi}{\delta\, x} + h = 0$, which is identical with Equation (IV).

It can also be seen that the displacement caused by the uniform flow does not enter into the equation; such flow, therefore, if it exists, can be treated separately from the oscillating motion.

4.—General Integral of Equations (*III*) *and* (*IV*).—As the motion for which an expression is desired is only that which is periodical, there must be taken the most general expression, depending on the time, which will satisfy the Differential Equation (III). Assume then

$$\xi = P \cos.\, \sigma\, t + Q \sin.\, \sigma\, t \ldots\ldots\ldots\ldots\ldots\ldots (1)$$

σ being an arbitrary constant, and P and Q functions of x to be discovered. Then

$$\frac{\delta\, \xi}{\delta\, t} = -\, \sigma\, P \sin.\, \sigma\, t + \sigma\, Q \cos.\, \sigma\, t$$

$$\frac{\delta^2\, \xi}{\delta\, t^2} = -\, \sigma^2\, P \cos.\, \sigma\, t - \sigma^2\, Q \sin.\, \sigma\, t$$

$$\frac{\delta^2\, \xi}{\delta\, x^2} = \frac{d^2\, P}{d\, x^2} \cos.\, \sigma\, t + \frac{d^2\, Q}{d\, x^2} \sin.\, \sigma\, t$$

Substituting in Equation (III), we find

$$-\, \sigma^2\, P \cos.\, \sigma\, t - \sigma^2\, Q \sin.\, \sigma\, t - f\, \sigma\, P \sin.\, \sigma\, t + f\, \sigma\, Q \cos.\, \sigma\, t$$
$$-\, g\, \gamma\, \frac{d^2\, P}{d\, x^2} \cos.\, \sigma\, t - g\, \gamma\, \frac{d^2\, Q}{d\, x^2} \sin.\, \sigma\, t = 0.$$

This equation must be satisfied for all values of t. Therefore, for $t = 0$ and $\sigma\, t = \frac{\eta}{2}$, the following equations must hold, respectively

$$\left.\begin{aligned} -\, \sigma^2\, P + f\, \sigma\, Q - g\, \gamma\, \frac{d^2\, P}{d\, x^2} = 0 \\ -\, \sigma^2\, Q - f\, \sigma\, P - g\, \gamma\, \frac{d^2\, Q}{d\, x^2} = 0 \end{aligned}\right\} \ldots\ldots\ldots\ldots (1_0)$$

Now let

$$P = A\, e^{\alpha\, x} \text{ and } Q = B\, e^{\alpha\, x} \quad \ldots\ldots\ldots\ldots\ldots\ldots (2)$$

then, substituting in Equation (1_0) and putting $x = 0$, the constants, A, B, α, will satisfy the equations

$$\left. \begin{array}{l} (\sigma^2 + g\,\gamma\,\alpha^2)\,A - f\,\sigma\,B = 0 \\ f\,\sigma\,A + (\sigma^2 + g\,\gamma\,\alpha^2)\,B = 0 \end{array} \right\} \ldots\ldots\ldots\ldots\ldots\ldots (3)$$

Eliminating the ratio, $\frac{A}{B}$, between Equations (3), we get

$$(\sigma^2 + g\,\gamma\,\alpha^2)^2 + f^2\,\sigma^2 = 0 \ldots\ldots\ldots\ldots\ldots\ldots (3_0)$$

which furnishes the characteristic equation

$$\alpha^2 = \frac{-\sigma^2 \pm f\,\sigma\,\sqrt{-1}}{g\,\gamma} \ldots\ldots\ldots\ldots\ldots\ldots (4)$$

from which

$$\alpha = p + q\,\sqrt{-1} \ldots\ldots\ldots\ldots\ldots\ldots (5)$$

and p and q are defined by

$$\left. \begin{array}{l} p^2 - q^2 = -\dfrac{\sigma^2}{g\,\gamma} \\ 2\,p\,q = \pm \dfrac{f\,\sigma}{g\,\gamma} \end{array} \right\} \ldots\ldots\ldots\ldots\ldots\ldots (6)$$

which give

$$\left. \begin{array}{l} p^2 = \dfrac{\sigma^2}{2\,g\,\gamma}\left[-1 + \sqrt{1 + \dfrac{f^2}{\sigma^2}}\right] \\ q^2 = \dfrac{\sigma^2}{2\,g\,\gamma}\left[1 + \sqrt{1 + \dfrac{f^2}{\sigma^2}}\right] \end{array} \right\} \ldots\ldots\ldots\ldots\ldots\ldots (7)$$

we also can write

$$\left. \begin{array}{l} p = \dfrac{\sigma}{\sqrt{2\,g\,\gamma}} \sqrt{-1 + \sqrt{1 + \dfrac{f^2}{\sigma^2}}} \\ q = \dfrac{\sigma}{\sqrt{2\,g\,\gamma}} \sqrt{1 + \sqrt{1 + \dfrac{f^2}{\sigma^2}}} \end{array} \right\} \ldots\ldots\ldots\ldots\ldots\ldots (8)$$

We get two solutions

$$\alpha = p + q\,\sqrt{-1} \text{ and } \alpha = -p + q\,\sqrt{-1} \ldots\ldots (9)$$

and two others by changing q to $-q$. The first value of α in Equation (9) gives $\alpha^2 = \dfrac{-\sigma^2 + f\,\sigma\,\sqrt{-1}}{g\,\gamma}$ and, with respect to Equation (3), $B = A\,\sqrt{-1}$. The second value of α in Equation (9) gives $\alpha^2 = \dfrac{-\sigma^2 - f\,\sigma\,\sqrt{-1}}{g\,\gamma}$, and, consequently, $B = -A\,\sqrt{-1}$. There-

fore, without regard to the constant, A, the first value of α in Equation (9) gives

$$P = e^{(p+q\sqrt{-1})x} \text{ and } Q = \sqrt{-1}\; e^{(p+q\sqrt{-1})x}$$

which can also be written

$$P = e^{px}\,(\cos.\, q\,x + \sqrt{-1}\,\sin.\, q\,x)$$
$$Q = e^{px}\,(-\sin.\, q\,x + \sqrt{-1}\,\cos.\, q\,x)$$

and from these the two solutions will be

$$\begin{cases} P = e^{px}\cos.\, q\,x & Q = -\,e^{px}\sin.\, q\,x \\ P = e^{px}\sin.\, q\,x & Q = e^{px}\cos.\, q\,x \end{cases}$$

The second value of α in Equation (9) gives

$$P = e^{-px}\,(\cos.\, q\,x + \sqrt{-1}\,\sin.\, q\,x)$$
$$Q = e^{-px}\,(\sin.\, q\,x - \sqrt{-1}\,\cos.\, q\,x)$$

and the two solutions

$$\begin{cases} P = e^{-px}\cos.\, q\,x & Q = e^{-px}\sin.\, q\,x \\ P = e^{-px}\sin.\, q\,x & Q = -\,e^{-px}\cos.\, q\,x \end{cases}$$

Adding these four solutions, multiplied by constants, the general solution of the factors, P and Q, of Equation (1) is obtained.

$$\left.\begin{aligned} P &= e^{px}\,(C\cos.\, q\,x + D\sin.\, q\,x) + e^{-px}\,(C'\cos.\, q\,x + D'\sin.\, q\,x) \\ Q &= e^{px}\,(-\,C\sin.\, q\,x + D\cos.\, q\,x) + e^{-px}\,(C'\sin.\, q\,x - D'\cos.\, q\,x) \end{aligned}\right\}\,..(10)$$

The value of α, obtained by changing q to $-\,q$, would give the same result.

Substituting these values of P and Q in Equation (1), the expression for the horizontal displacement will read

$$\begin{aligned} \xi = {} & e^{px}\,[C\cos.\,(\sigma\,t + q\,x) + D\sin.\,(\sigma\,t + q\,x)] \\ & + e^{-px}\,[C'\cos.\,(\sigma\,t - q\,x) - D'\sin.\,(\sigma\,t - q\,x)]\,..(11) \end{aligned}$$

To every expression found for ξ another corresponds for the height, h, and, by Equation (IV),

$$\begin{aligned} h = {} & -\,\gamma\,e^{px}\,[(C\,p + D\,q)\cos.\,(\sigma\,t + q\,x) + (D\,p - C\,q)\sin.\,(\sigma\,t + q\,x)] \\ & + \gamma\,e^{-px}\,[(C'p - D'q)\cos.\,(\sigma\,t - q\,x) + (D'\,p + C'q)\sin.\,(\sigma\,t - q\,x)]\,..(12) \end{aligned}$$

The numerical values of the constants depend on the particular conditions of the problem.

5.—Determination of the Constants in a Canal Without Proper Tide, Communicating at One End with a Tideless Lake and at the

Other End with a Tidal Sea.—Instead of the constants, C, D, C', D', of Equations (*11*) and (*12*), let there be introduced four new constants, A, B, A', B', being related to the former as follows:

$$C\,p + D\,q = \frac{A}{\gamma} \qquad C'\,p - D'\,q = \frac{A'}{\gamma}$$

$$-\,C\,q + D\,p = \frac{B}{\gamma} \qquad C'\,q + D'\,p = \frac{B'}{\gamma}$$

and, therefore,

$$\left.\begin{aligned} C &= \frac{p\,A - q\,B}{\gamma\,(p^2 + q^2)} & D &= \frac{q\,A + p\,B}{\gamma\,(p^2 + q^2)} \\ C' &= \frac{p\,A' + q\,B'}{\gamma\,(p^2 + q^2)} & D' &= \frac{-\,q\,A' + p\,B'}{\gamma\,(p^2 + q^2)} \end{aligned}\right\} \ldots\ldots (13)$$

Consequently, Equations (*11*) and (*12*) will become

$$\xi = \frac{1}{\gamma\,(p^2 + q^2)} \left\{ \begin{aligned} & e^{px}\,[(p\,A - q\,B) \cos.\,(\sigma\,t + q\,x) + (q\,A + p\,B) \\ & \sin.\,(\sigma\,t + q\,x)] + e^{-px}\,[(p\,A' + q\,B') \\ & \cos.\,(\sigma\,t - q\,x) + (q\,A' - p\,B') \\ & \sin.\,(\sigma\,t - q\,x)] \end{aligned} \right\} \ldots (14)$$

$$\begin{aligned} h = {} & e^{px}\,[-\,A \cos.\,(\sigma\,t + q\,x) - B \sin.\,(\sigma\,t + q\,x)] \\ & + e^{-px}\,[A' \cos.\,(\sigma\,t - q\,x) - B' \sin.\,(\sigma\,t - q\,x)] \ldots\ldots (15) \end{aligned}$$

Let the origin of the abscissas, x, be at the lake end of the canal. Then we must have

$$\text{For } x = 0,\ h = 0 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (o)$$

$$\text{For } x = L,\ h = h_0 \sin.\,\sigma\,t \ldots\ldots\ldots\ldots\ldots\ldots (r)$$

where L = the length of the canal, h_0 the given half amplitude of the sinoidal tide, and $\sigma = \frac{2\,\pi}{T}$, if T is the interval between consecutive high tides. The origin of times is taken at $\frac{T}{4}$ preceding high water at the sea end.

To determine the constants, it is known from the condition in Equation (*o*) that

$$A' - A = 0 \text{ and } B' + B = 0,$$

and from the condition in Equation (*r*) that

$$e^{p\,L}\,(-\,A \cos.\,q\,L - B \sin.\,q\,L) + e^{-p\,L}\,(A' \cos.\,q\,L + B' \sin.\,q\,L) = 0.$$

$$e^{p\,L}\,(A \sin.\,q\,L - B \cos.\,q\,L) + e^{-p\,L}\,(A' \sin.\,q\,L - B' \cos.\,q\,L) = h_0.$$

If there be put, for convenience,

$$\Delta = 2 \left(\frac{e^{2pL} + e^{-2pL}}{2} - \cos. 2qL \right) \qquad (16)$$

then

$$\left. \begin{aligned} A = A' &= \frac{h_0}{\Delta} (e^{pL} + e^{-pL}) \sin. qL \\ B = -B' &= -\frac{h_0}{\Delta} (e^{pL} - e^{-pL}) \cos. qL \end{aligned} \right\} \qquad (17)$$

and, substituting these values in Equations (14) and (15),

$$\xi = \frac{h_0}{\gamma \Delta (p^2 + q^2)} \left\{ \begin{aligned} & p \left\{ \begin{aligned} & - e^{p(L+x)} \sin. [\sigma t - q (L - x)] \\ & + e^{-p(L-x)} \sin. [\sigma t + q (L + x)] \\ & - e^{p(L+x)} \sin. [\sigma t - q (L + x)] \\ & + e^{-p(L-x)} \sin. [\sigma t + q (L - x)] \end{aligned} \right\} \\ & + q \left\{ \begin{aligned} & e^{p(L+x)} \cos. [\sigma t - q (L - x)] \\ & - e^{-p(L-x)} \cos. [\sigma t + q (L + x)] \\ & + e^{p(L+x)} \cos. [\sigma t - q (L + x)] \\ & - e^{-p(L-x)} \cos. [\sigma t + q (L - x)] \end{aligned} \right\} \end{aligned} \right\} \qquad (18)$$

$$h = \frac{h_0}{\Delta} \left\{ \begin{aligned} & e^{p(L+x)} \sin. [\sigma t - q (L - x)] \\ & - e^{-p(L-x)} \sin. [\sigma t + q (L + x)] \\ & - e^{p(L-x)} \sin. [\sigma t - q (L + x)] \\ & + e^{-p(L+x)} \sin. [\sigma t + q (L - x)] \end{aligned} \right\} \qquad (19)$$

The current will have the expression

$$\frac{\delta \xi}{\delta t} = \frac{h_0}{\gamma \Delta (p^2 + q^2)} \left\{ \begin{aligned} & p \left\{ \begin{aligned} & - e^{p(L+x)} \cos. [\sigma t - q (L - x)] \\ & + e^{-p(L-x)} \cos. [\sigma t + q (L + x)] \\ & - e^{p(L+x)} \cos. [\sigma t - q (L + x)] \\ & + e^{-p(L-x)} \cos. [\sigma t + q (L - x)] \end{aligned} \right\} \\ & + q \left\{ \begin{aligned} & - e^{p(L+x)} \sin. [\sigma t - q (L - x)] \\ & + e^{-p(L-x)} \sin. [\sigma t + q (L + x)] \\ & - e^{p(L+x)} \sin. [\sigma t - q (L + x)] \\ & + e^{-p(L-x)} \sin. [\sigma t + q (L - x)] \end{aligned} \right\} \end{aligned} \right\} \qquad (20)$$

These equations show that the motion in the canal consists of the superposition of four waves, two of which are propagated from the sea toward the lake, having a speed of $-\frac{\sigma}{q}$, and two in the opposite

direction, with a speed of $+\frac{\sigma}{q}$. The absolute value of these speeds, according to Equation (8) is

$$\frac{\sigma}{q}=\sqrt{\frac{2\,g\,\gamma}{1+\sqrt{1+\frac{f^2}{\sigma^2}}}}\quad\text{.................}(21)$$

which, if friction is not considered, reduces to the well-known value of $\frac{\sigma}{q}=\sqrt{g\,\gamma}$.

The four waves, which all have the same period, $\frac{2\,\pi}{\sigma}=T$, as the tidal sea, can be united into one, the height of which, h, is given by the equation:

$$h=P\cos.\,\sigma\,t+Q\sin.\,\sigma\,t\quad\text{.................}(22)$$

or, putting $\frac{P}{Q}=\tan.\,Z$, there can also be written

$$h=\sqrt{P^2+Q^2}\,\sin.\,(\sigma\,t+Z)\quad\text{.................}(23)$$

In this equation

$$\left.\begin{aligned}P&=\frac{h_0}{\Delta}\left\{\begin{aligned}&-e^{p\,(L+x)}\sin.\,q\,(L-x)\\&-e^{-p\,(L-x)}\sin.\,q\,(L+x)\\&+e^{p\,(L-x)}\sin.\,q\,(L+x)\\&+e^{-p\,(L+x)}\sin.\,q\,(L-x)\end{aligned}\right\}\\Q&=\frac{h_0}{\Delta}\left\{\begin{aligned}&e^{p\,(L+x)}\cos.\,q\,(L-x)\\&-e^{-p\,(L-x)}\cos.\,q\,(L+x)\\&-e^{p\,(L-x)}\cos.\,q\,(L+x)\\&+e^{-p\,(L+x)}\cos.\,q\,(L-x)\end{aligned}\right\}\end{aligned}\right\}\quad\text{........}(24)$$

If we substitute the hyperbolic functions,

$\sin._h\,x=\frac{e^x-e^{-x}}{2}$, $\cos._h\,x=\frac{e^x+e^{-x}}{2}$, $\tan._h\,x=\frac{\sin._h\,x}{\cos._h\,x}$, then

$$\left.\begin{aligned}P&=\frac{2\,h_0}{\Delta}\left\{\begin{aligned}&[\sin._h\,p\,(L-x)\sin.\,q\,(L+x)\\&\quad-\sin._h\,p\,(L+x)\sin.\,q\,(L-x)]\end{aligned}\right\}\\Q&=\frac{2\,h_0}{\Delta}\left\{\begin{aligned}&[-\cos._h\,p\,(L-x)\cos.\,q\,(L+x)\\&\quad+\cos._h\,p\,(L+x)\cos.\,q\,(L-x)]\end{aligned}\right\}\end{aligned}\right\}\quad\text{......}(25)$$

The condition for the occurrence of maximum elevation at the section, x, is evidently that $\sin.\,(\sigma\,t+Z)=1$; for this case

$$h_{max.}=h_0\sqrt{\frac{\cos._h\,2\,p\,x-\cos.\,2\,q\,x}{\cos._h\,2\,p\,L-\cos.\,2\,q\,L}}\quad\text{...........}(26)$$

The time of this maximum will be found from the condition $\sigma t + Z = \frac{\pi}{2}$

$$t = \frac{\pi}{2\sigma} - \frac{Z}{\sigma} = \frac{T}{4} - \frac{Z}{2\pi} T \ldots\ldots\ldots\ldots\ldots\ldots (27)$$

or, in other words, high water at the section, x, will occur $-\frac{Z}{2\pi} T$ later than at the sea.

The value of $Z = \frac{P}{Q}$, with respect to Equations (*16*), (*23*), and (*25*), which also furnished Equation (*26*), will be found from

$$\tan. Z = \frac{-\tan. q L \tan._h p x + \tan._h p L \tan. q x}{\tan._h p L \tan._h p x + \tan. q L \tan. q x} \ldots\ldots (28)$$

The expression of the current velocity also can be written in the form

$$\frac{\delta \xi}{\delta t} = M \cos. \sigma t + N \sin. \sigma t = \sqrt{M^2 + N^2} \sin. (\sigma t + Y) \ldots (29)$$

where $\tan. Y = \frac{M}{N}$, and M and N are given as follows:

$$\left.\begin{array}{l} M = \dfrac{2 h_0 \sigma}{\gamma \Delta (p^2 + q^2)} \left\{ \begin{array}{l} p\,[\sin._h p\,(L + x) \cos. q\,(L - x) \\ + \sin._h p\,(L - x) \cos. q\,(L + x)] \\ + q\,[-\cos._h p\,(L + x) \sin. q\,(L - x) \\ - \cos._h p\,(L - x) \sin. q\,(L + x)] \end{array} \right\} \\ N = \dfrac{2 h_0 \sigma}{\gamma \Delta (p^2 + q^2)} \left\{ \begin{array}{l} p\,[\cos._h p\,(L + x) \sin. q\,(L - x) \\ + \cos._h p\,(L - x) \sin. q\,(L + x)] \\ + q\,[\sin._h p\,(L + x) \cos. q\,(L - x) \\ + \sin._h p\,(L - x) \cos. q\,(L + x)] \end{array} \right\} \end{array} \right\} \ldots (30)$$

The value of the maximum current will be $= \sqrt{M^2 + N^2}$, and will occur when $\sin. (\sigma t + Y) = 1$, or $\sigma t + Y = \frac{\pi}{2}$,

and $$t = \frac{T}{4} - \frac{Y}{2\pi} T \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (31)$$

6.—General Remarks.—The theory evolved in the preceding paragraphs and the equations presented give a comparatively easy and, for all practical purposes, correct solution of the problem of tidal phenomena in canals, if the basic assumptions are approximated in practice.

The tidal variations at the end of the canal can generally be represented very closely by an equation having the following form:

$$h = h_0 \sin. \sigma t + h_0' \sin. \sigma' t' + \ldots\ldots\ldots\ldots\ldots\ldots (32)$$

The equations in this paper furnish the solution for each member of the right-hand side of Equation (*32*), and these results, added together by the principle of superposition of waves, will solve the problem. If both ends of the canal are subjected to tidal influences, they should be treated separately and the results added.

If there is a difference between the mean elevations of the two seas, the influence of a uniform motion resulting from this circumstance should be added to the results obtained for the oscillating motion. If a_0 = the elevation of mean sea level at one end of the canal, and b_0 = that at the other end, then at any point of the canal, characterized by the abscissa, x, the elevation of the water, $a = a_0 - \frac{a_0 - b_0}{L} x$, and the value of the uniform velocity can be expressed approximately by the equation, $v = \frac{g I}{f}$, where $I = \frac{a_0 - b_0}{L}$, or by any equation derived for uniform flow in canals. If the canal is not very long, and the difference in mean sea levels is considerable, the equations for non-uniform but permanent flow should be applied. Later, in this paper, these equations will be dealt with in some detail.

In deriving the equations the assumption was made that the friction, F, is proportional to the first power of the velocity of the flow. Strictly speaking, therefore, they can be applied with precision only on canals of great length subjected to small differences in head, that is, in which the velocities are small enough to make the condition prevail. The integration of the Differential Equation (I) in cases where the power, n, of the velocity to which the frictional resistance is proportional differs from unity still awaits analytical solution.

To the writer's knowledge, no attempt has been made as yet to compensate for this deficiency and to amend the equations in such a way that they could be used for canals of the class of the Cape Cod Canal, in which the velocity is such that F is proportional to about the square of the velocity. This is probably due to the fact that no actual case has arisen, up to the present time, where any proposed theory could be substantiated by the results of careful measurements, and that for all tidal canals in existence, approximate calculations by equations based on Bernoulli's theorem give velocities sufficiently close for practical purposes.

In an entirely different field of hydrodynamics, however, it was imperative to evaluate the influence of the frictional resistances, when

proportional to the square of the velocity, on the oscillating motion of fluids. With the advent of long pipe lines supplying hydraulic power plants, means had to be found for the proper regulation of the wheels and for the protection of the conduits. The surge-tank regulator was devised for this purpose, and its correct design necessitated the prediction of the magnitude of the rise and fall of the water in it for the assumed operating conditions.

Within the last 8 or 10 years a number of books and papers have been published on this subject, mostly from the pens of American and Swiss engineers and scientists.* It can be stated that the problem of the surge tank is, to an extent, the inverse of the problem of the current in a tidal canal. In the former, from known ultimate velocity changes in the conduit, the oscillating motion induced within the reservoir at the end of the pipe is sought; and in the latter, the velocity changes in the canal due to the known harmonic changes of the head at the end are investigated.

The differential equation characterizing the motions of the water in the surge tank is, as could be inferred from the foregoing considerations, similar to Equation (*I*) in this paper, and the investigators of the surges were confronted by the same difficulty, namely, the impossibility of the analytical integration of the term, $\int v^2 \, ds$. The evaluation of this integral, however, was absolutely necessary, in view of the more and more frequent application of the surge-tank regulator, and the question was attacked in both a theoretical and practical way. I. P. Church, Assoc. Am. Soc. C. E., in discussing Mr. Johnson's† and Mr. Warren's‡ papers, has shown that this expression can be integrated graphically and results derived with a great degree of accuracy. Other writers, as Dr. Prasil and Robert Dubs,§ have shown that the tedious graphical method can be avoided and excellent results obtained by using an average value of

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* "Wasserschloss Probleme," by Professor F. Prasil, *Schweizerische Bauzeitung*, Vol. LII, No. 21 and following.

"Allgemeine Theorie über die veränderliche Bewegung des Wassers in Leitungen," by Robert Dubs and V. Bataillard, Berlin, 1909.

"The Surge Tank in Water Power Plants," by R. D. Johnson, *Transactions*, Am. Soc. Mech. Engrs., Vol. 30, 1908.

"The Differential Surge Tank," by R. D. Johnson, *Transactions*, Am. Soc. C. E., Vol. LXXVIII, 1915.

"Penstock and Surge-Tank Problems," by Minton M. Warren, *Transactions*, Am. Soc. C. E., Vol. LXXIX, 1915.

† *Transactions*, Am. Soc. Mech. Engrs., Vol. 30, p. 488 and following.

‡ *Transactions*, Am. Soc. C. E., Vol. LXXIX, p. 273 and following.

§ "Allgemeine Theorie," etc., II Teil: Stollen und Wasserschloss, pp. 219-221.

the velocity, such that the frictional force, as a linear function of the same, will absorb the same total energy in the interval during which the velocity of the conduit changes from zero to maximum, as the total energy absorbed by the frictional force in the same interval, regarding F as proportional to the square of the velocity at any instant. This simplified method is endorsed by Mr. Johnson,* and a great number of experiments have proved the reliability of its use.

On the strength of the foregoing argument, the writer proposes the following modification of Equations (c) and (III) and those deduced therefrom.

In the case of variable motion, where the velocity ranges from zero to a considerable value, as in the Cape Cod Canal, for the value of f_0, as given in textbooks on hydraulics for different consistency of the wetted perimeter, a different value, f_0', should be substituted, so that the equation

$$F_{average} = f_0' \, v_{mean} \ldots\ldots\ldots\ldots\ldots\ldots\ldots (c_1)$$

will be satisfied.

For streams of the description of the Cape Cod Canal, $n = 2$ in the equation, $F = f_0 \, v^n$. Now, $F = f_0 v^2$ can be represented by the ordinates of a parabola, the average ordinate of which is 75% of the maximum. Due to the fact disclosed by more recent investigations that f_0 itself is not a constant quantity but varies slightly inversely with the velocity, it is proposed to use the value, $F_{average} = 0.7 \, F_{max.}$, and therefore $f_0' = \dfrac{0.7 \, F_{max.}}{0.5 \, v_{max.}}$, $0.5 \, v_{max.}$ being the mean velocity. The value of $v_{max.}$, to be used in such cases, can be established by any of the approximate methods to be described later. For maximum velocities less than 1.5 ft., the original assumption can be considered as correct, and the use of f_0 as given for 1 ft. velocity is recommended, it being a well-known fact that at such low velocities the friction is actually proportional to the velocity.

If the maximum velocity computed by the submitted equations would considerably differ from the velocity given by the approximate methods, the value of f_0' should be corrected for the former, and the whole calculation refigured with this new value of f_0'. This process may be repeated until a satisfactory agreement between the figured value of $v_{max.}$ and that of f_0' is reached.

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* *Transactions*, Am. Soc. Mech. Engrs., Vol. 30, p. 455.

Mr. Lévy, in applying the equations to the Suez Canal, used a coefficient, $\frac{f_0}{\eta} = 0.0005$, and found that the computed are far below the actual velocities, and therefore suggested the use of equations based on the extension of Equation (*III*) to a second approximation. Judging from the close check of the computed and observed velocities of the Cape Cod Canal, the equations herein presented give reliable results for all practical purposes, if the value of f_0' is taken in accordance with the previous reasoning. In the case of the Suez Canal, for the conditions considered by Mr. Lévy, the approximate methods furnish a $v_{max.}$ of about 2.5 ft. For the consistency of the Suez Canal bed, f_0 can be taken as 0.004. Then

$$f_0' = \frac{0.7 \times 0.004 \times 2.5^2}{1.25} = 0.014, \text{ and } \frac{f_0'}{\eta} = 0.00022,$$

which value gives velocities very close to the observed results in the Suez Canal.

Dr. Rollin A. Harris* gives the following values of f_0 for different consistencies of the wetted perimeter:

Fine sand	0.00405	These values are called the Eytelwein frictional values.
Coarse sand	0.00488	
Beds of streams	0.00756	

7.—*Application of the Theory to the Cape Cod Canal.*—In order to test the validity of the equations supposed to furnish the values for the elevation of the water and velocity at any section and time, the equations must be applied to a stretch of uniform or nearly uniform cross-section. Such a stretch, according to the general plan, Fig. 2, extends from about Station 74 to Station 382. The nearest points, where observations were taken on August 26th, 1915, are Station 80 and Station 375, which we will consider as the ends of the canal having a uniform bottom width of 100 ft., so that we will make the length of the canal $L = 29\,500$ ft.

The mareographs of these stations, taken from the observations, are shown on Fig. 32, together with the sine curves having the same periods and amplitudes, and which are considered in the computations as being the tidal variations affecting the canal. It will be noted that the sine curve for Station 80, drawn with mean sea level at Elevation 100, is very close to the actual curve for the whole range

* "Manual of Tides," Part V., p. 249.

of the observation; but, at Station 375, though Elevation 100 divides the period into two almost equal portions, the second part of the cycle has a very much smaller amplitude than the first. This is typical for the westerly end, for reasons previously explained.

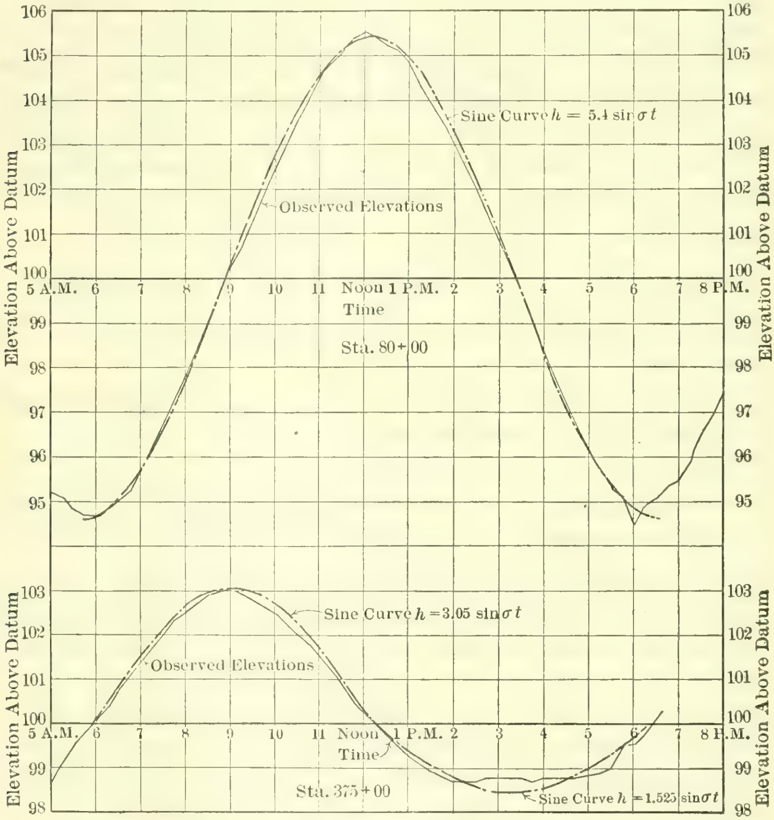

FIG. 32.

The bottom of the canal is at Elevation 70.35 at Station 80 and at Elevation 72.35 at Station 375, having a practically uniform slope. The reduced depths of the two end sections differ only by about 3% from the reduced depth based on a mean section having the bottom at Elevation 71.35, and therefore it may be considered justifiable to use this mean section (see Fig. 33*) as applying to the whole canal, instead of going into the exceedingly more complicated question of

* Fig. 33 is not drawn to scale.

solving the tidal motions in a canal of variable depth. It is evident that, with the degree of approximation permitted throughout in the derivation of the equations, this substitution is permissible.

It is also assumed that the direct effect of the moon and sun on the water in the canal is negligible.

The bed of the canal consists of coarse gravel and boulders, and the broken-stone slope protection tends to make the wetted perimeter still rougher. Considerable energy is expended by the water in scouring the bottom and transporting the material. In other words, the canal comes under the same group as ordinary streams, and the selection of the corresponding coefficient $f_0 = 0.00756$, seems to be justified. The same coefficient was adopted by the officers of the United States Coast and Geodetic Survey when asked to predetermine the velocities in the Cape Cod Canal. In approximate calculations, referred to later, it has been found that, for conditions prevailing at the time of the observations on August 26th, 1915, the maximum velocity in the canal is not expected to exceed 5 ft. per sec. Therefore,

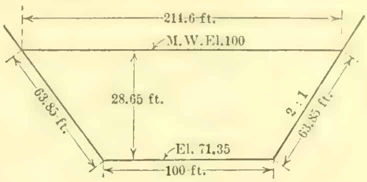

FIG. 33.

$$F_{max.} = f_0\, v^2 = 0.00756 \times 5^2 = 0.189 \text{ lb. per sq. ft.}$$

$$f_0' = \frac{0.7\, F_{max.}}{0.5\, v_{max.}} = \frac{0.7 \times 0.189}{2.5} = 0.528 \text{ lb. per cu. ft.}$$

In the following will be presented the numerical calculations for establishing the velocities at Stations 80, 227 + 50, and 375, and the water elevations at Station 227 + 50, on the basis of the tidal observations of August 26th, 1915. These calculations are divided into four parts. Under (*A*) the determinations of the constant quantities entering into the computations are given; under (*B*) and (*C*) the influences of the tides at the two ends are treated separately; under (*D*) the superposition of the water elevations and current velocities, as found under (*B*) and (*C*), is shown graphically.

(*A*).—*Determination of Constant Quantities.*—

g = acceleration of gravity........................ 32.2 ft.

π = weight of 1 cu. ft. of sea water.............. 64.0 lb.

Z = wetted perimeter of undisturbed canal........ 277.7 ft.

Ω = area of undisturbed cross-section............. 4 510.0 sq. ft.

R = hydraulic radius.............................. 19.8 ft.

γ = reduced depth.............................. 21.0 ft.

L = length of canal..............................29 500.0 ft.

T = interval between consecutive high tides:

Station 80........................12 hr. 54 min.

Station 375........................12 hr. 24 min.

$$\sigma = \frac{2\pi}{T} = 0.00014 \text{ radian per second;}$$

$$f_0 = 0.00756,\ f_0' = 0.0528,\ \frac{f_0'}{\eta} = 0.000825;$$

$$f = g \times \frac{f_0'}{\eta} \times \frac{1}{R} = 0.00134;$$

$$\frac{f}{\sigma} = 9.57,\ \frac{f^2}{\sigma^2} = 91.6;$$

$$p = \frac{\sigma}{\sqrt{2\,g\,\gamma}}\sqrt{-1 + \sqrt{1 + 91.6}} = 0.0000111;$$

$$q = \frac{\sigma}{\sqrt{2\,g\,\gamma}}\sqrt{1 + \sqrt{1 + 91.6}} = 0.0000124;$$

$$\sqrt{p^2 + q^2} = 0.0000167,\ \Delta = 2\,(\cos._h\, 2\,p\,L - \cos.\, 2\,q\,L)$$

$$= 0.96,\ \sqrt{2\,g\,\gamma} = 36.75$$

TABLE 15.

		arc.	sin.	cos.	tan.	$\sin._h$	$\cos_h$	$\tan._h$
For $x = L$ = 29 500 ft.	$p\,L = 0.328$	18° 50′	0.323	0.946	0.341	0.334	1.054	0.317
	$q\,L = 0.366$	21° 00′	0.358	0.934	0.384	0.375	1.068	0.351
	$2\,p\,L = 0.656$	37° 40′	0.611	0.792	0.772	0.704	1.223	0.576
	$2\,q\,L = 0.732$	42° 00′	0.669	0.743	0.900	0.800	1.280	0.624
For x and $(L - x)$ = 14 750 ft.	$p\,x = 0.164$	9° 25′	0.164	0.986	0.166	0.165	1.013	0.163
	$q\,x = 0.183$	10° 30′	0.182	0.983	0.185	0.184	1.017	0.181
	$2\,p\,x = 0.328$	18° 50′	0.323	0.946	0.341	0.334	1.054	0.317
	$2\,q\,x = 0.366$	21° 00′	0.358	0.934	0.384	0.375	1.068	0.351
For $(L + x)$ = 44 250 ft.	$p\,(L + x) = 0.492$	28° 15′	0.473	0.881	0.537	0.512	1.124	0.456
	$q\,(L + x) = 0.549$	31° 30′	0.523	0.853	0.613	0.577	1.155	0.500
	$2\,p\,(L + x) = 0.984$	56° 30′	0.834	0.552	1.511	1.151	1.524	0.755
	$2\,q\,(L + x) = 1.098$	63° 00′	0.891	0.454	1.963	1.332	1.667	0.800

(B).—Height of Tide, Current Velocities, and Retardations in the Canal, Due to Tidal Variations at Station 80.—

h_0 = half amplitude = 5.4 ft.

(a) At Station 80.................$x = L$ = 29 500 ft.

The elevation of the water at any time must be the same as the mareograph at Station 80, and is given by $h = h_0$ sin. $\sigma\, t$, where the zero time is $\frac{T}{4}$ before high water.

The current velocity:

$v = \sqrt{M^2 + N^2}$ sin. $(\sigma\, t + Y)$ according to Equation (29), where from Equation (30),

$$M = \frac{2\, h_0\, \sigma}{\gamma\, \Delta\, (p^2 + q^2)} (-\, q \text{ sin. } 2\, q\, L + p \text{ sin.}_h\, 2\, p\, L).$$

$$N = \frac{2\, h_0\, \sigma}{\gamma\, \Delta\, (p^2 + q^2)} (p \text{ sin. } 2\, q\, L + q \text{ sin.}_h\, 2\, p\, L)$$

$$v_{max.} = \sqrt{M^2 + N^2} = \frac{2\, h_0\, \sigma}{\gamma\, \Delta \sqrt{p^2 + q^2}} \sqrt{\text{sin.}^2\, 2\, q\, L + \text{sin.}_h^2\, 2\, p\, L}$$

$$= \frac{2 \times 5.4 \times 0.00014}{21 \times 0.96 \times 0.0000167} \sqrt{0.669^2 + 0.704^2}$$

$$= 4.35 \text{ ft. per sec.}$$

$$\text{tan. } Y = \frac{M}{N} = \frac{-\, q \text{ sin. } 2\, q\, L + p \text{ sin.}_h\, 2\, p\, L}{p \text{ sin. } 2\, q\, L + q \text{ sin.}_h\, 2\, p\, L}$$

$$= \frac{-\, 0.0000124 \times 0.669 + 0.0000111 \times 0.704}{0.0000111 \times 0.669 + 0.0000124 \times 0.704} = -\, 0.0303$$

or $Y = -\, 1° \; 45'$

$v_{max.}$ will occur, when $t = \frac{T}{4} - \frac{Y}{\sigma}$, and the retardation equals

$$\frac{1°45'}{360°} \times 46\,440 = 226 \text{ sec., or about 4 min.}$$

The maximum velocity, therefore, is retarded by 4 min. after high water.

(*b*) At Station 227+50..........$x = 14\,750$ ft.

The maximum elevation of the water is given by Equation (*26*)

$$h_{max.} = h_0 \sqrt{\frac{\text{cos.}_h\, 2\, p\, x - \text{cos. } 2\, q\, x}{\text{cos.}_h\, 2\, p\, L - \text{cos. } 2\, q\, L}} = 5.4 \times 0.5 = 2.70 \text{ ft.}$$

The time of this maximum will be found by Equation (*28*),

$$\text{tan. } Z = \frac{-\, \text{tan. } q\, L \text{ tan.}_h\, p\, x + \text{tan.}_h\, p\, L \text{ tan. } q\, x}{\text{tan.}_h\, p\, L \text{ tan.}_h\, p\, x + \text{tan. } q\, L \text{ tan. } q\, x}$$

$$= \frac{-\, 0.384 \times 0.163 + 0.317 \times 0.185}{0.317 \times 0.163 + 0.384 \times 0.185} = -\, 0.0322, \text{ and } Z = -\, 1°50'.$$

The retardation, therefore, $= \frac{1°\; 50'}{360°} \times 46\,440$, or about 4 min.

The current velocity:

Again, $v = \sqrt{M^2 + N^2} \sin. (\sigma t + Y)$, where, from Equation (*30*), denoting

$$\sin._h p (L + x) \cos. q (L - x) \text{ by } a = 0.512 \times 0.983 = 0.503$$
$$\sin._h p (L - x) \cos. q (L + x) \text{ by } b = 0.165 \times 0.853 = 0.141$$
$$\cos._h p (L + x) \sin. q (L - x) \text{ by } c = 1.124 \times 0.182 = 0.205$$
$$\cos._h p (L - x) \sin. q (L + x) \text{ by } d = 1.013 \times 0.523 = 0.530$$

we can write

$$M = \frac{2 h_0 \sigma}{\gamma \Delta (p^2 + q^2)} [p (a + b) - q (c + d)],$$

$$N = \frac{2 h_0 \sigma}{\gamma \Delta (p^2 + q^2)} [p (c + d) + q (a + b)]$$

$$v_{max.} = \sqrt{M^2 + N^2} = \frac{2 h_0 \sigma}{\gamma \Delta \sqrt{p^2 + q^2}} \sqrt{(a + b)^2 + (c + d)^2}$$

$$= \frac{2 \times 5.4 \times 0.00014}{21 \times 0.96 \times 0.0000167} \sqrt{0.644^2 + 0.735^2} = 4.4 \text{ ft. per sec.}$$

$$\tan. Y = \frac{M}{N} = \frac{p (a + b) - q (c + d)}{p (c + d) + q (a + b)}$$

$$= \frac{0.0000111 \times 0.644 - 0.0000124 \times 0.735}{0.0000111 \times 0.735 + 0.0000124 \times 0.644} = - 0.122,$$

and $$Y = - 7°.$$

The retardation equals $\frac{7°}{360°} \times 46\,440 = 910$ sec., or about 15 min.

(*c*) At Station 375............$x = 0$.

The elevation of the water is given by $h = 0$.

The current velocity:

$$v_{max.} = \frac{4 h_0 \sigma}{\gamma \Delta \sqrt{p^2 + q^2}} \sqrt{\cos.^2 q L \sin._h^2 p L + \sin.^2 q L \cos._h^2 p L}$$

$$= \frac{4 \times 5.4 \times 0.00014}{21 \times 0.96 \times 0.0000167} \sqrt{0.934^2 \times 0.334^2 + 0.358^2 \times 1.054^2}$$

$$= 4.31 \text{ ft. per sec.}$$

$$\tan. Y = \frac{p \cos. q L \sin._h p L - q \sin. q L \cos._h p L}{p \sin. q L \cos._h p L + q \cos. q L \sin._h p L}$$

$$= \frac{1.0000111 \times 0.934 \times 0.334 - 0.0000124 \times 0.358 \times 1.054}{0.0000111 \times 0.358 \times 1.054 + 0.0000124 \times 0.934 \times 0.334}$$

$$= - 0.15, \text{ and } Y = 8° 30'.$$

The retardation, therefore, will be $\frac{8° 30'}{360°} \times 46\,440 = 1\,100$ sec., or about 18 min.

(*C*).—*Influence of the Tide at Station 375.*—

h_0 = half amplitude for first part of cycle = 3.05 ft.
h'_0 = " " " second " " " = 1.525 ft.

It is evident, by inspection of the equations, that the results found under (*B*) for retardations will be the same. For the height of water and the velocities, the corresponding figures should be multiplied by the ratio, $\frac{3.05}{5.4}$, for the first half, and by $\frac{1.525}{5.4}$ for the second half, of the cycle.

(*a*) At Station 80........................ $x = 0$.

The elevation of the water is given by $h = 0$.

The maximum velocity will be

$$4.31 \times \frac{3.05}{5.4} = 2.43 \text{ ft. per sec. for the first half}$$

and $4.31 \times \frac{1.525}{5.4} = 1.22$ ft. per sec. for the second half of the cycle.

The retardation of the maximum velocity will be 18 min.

(*b*) At Station 227 + 50...................... $x = 14\,750$ ft.

The maximum water elevation equals $2.70 \times \frac{3.05}{5.4} = 1.53$ ft. and $2.70 \times \frac{1.525}{5.4} = 0.77$ ft., respectively.

The retardation is 4 min.

The maximum velocity equals $4.4 \times \frac{3.05}{5.4} = 2.49$ ft. per sec. and $4.4 \times \frac{1.525}{5.4} = 1.25$ ft. per sec., respectively.

The retardation is 15 min.

(*c*) At Station 375...................... $x = L = 29\,500$ ft.

The elevation of the water is given by $h = h_0 \sin. \sigma t$.

The maximum velocity will be $4.35 \times \frac{3.05}{5.4} = 2.46$ ft. per sec. and $4.35 \times \frac{1.525}{5.4} = 1.23$ ft. per sec., respectively.

The retardation is 4 min.

(*D*).—*Combination of* (*B*) *and* (*C*).—Figs. 34, 35, and 36 show the velocities at Stations 80, 227 + 50, and 375, and Fig. 37 shows the water elevation at Station 227 + 50, all platted as ordinates to the

times, which are considered as the abscissas of the curves. The curves in full lines show the observed velocities reduced to mean velocities; the curves in dotted lines show the component velocities plotted as sine curves to the computed maximum values, which have been located according to the figured retardations; and the curves in lines of dashes and dots the resulting velocities, the components having been added graphically. The same method has been followed in preparing Fig. 37.

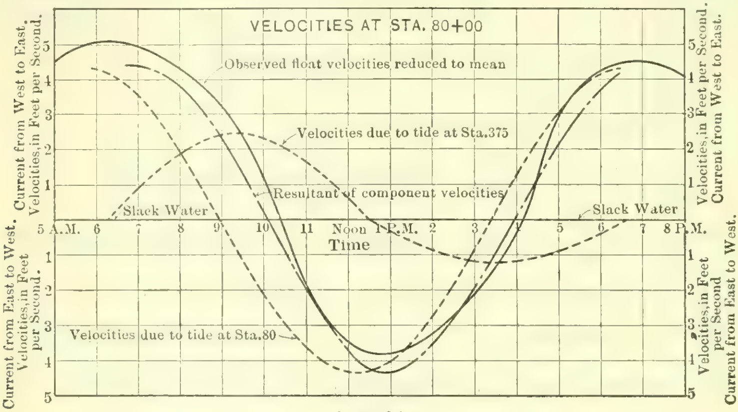

FIG. 34.

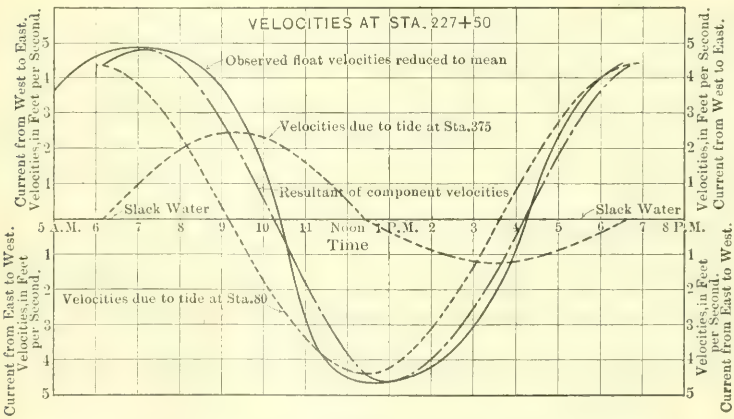

FIG. 35.

It should be noted that no observations were taken at Station 227 + 50, but they were made at Station 225, only 250 ft. distant. The latter have been taken for the sake of comparison.

The cross-section of the canal at Station 375 is actually very much larger than designed, the area below mean sea level being 5 900 sq. ft.

In platting the results for this station, therefore, the computed velocities have been multiplied by the ratio, $\frac{4\ 510}{5\ 900} = 0.765$.

8.—Results.—Inspection of Figs. 34, 35, 36, and 37 shows a remarkably close check between computed and observed conditions, in so far as the water elevations, maximum velocity of the current, and the time of maximum current and slack water are concerned. The greatest deviations for maximum velocities occur at Station 80, where they amount to more than 10 per cent. They are somewhat smaller at Station 375, and the results check almost exactly at Station 227 + 50.

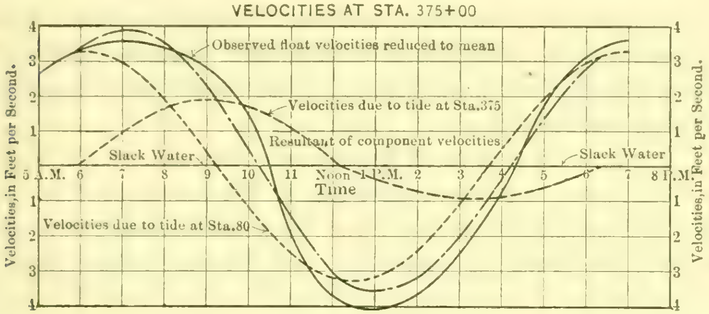

FIG. 36.

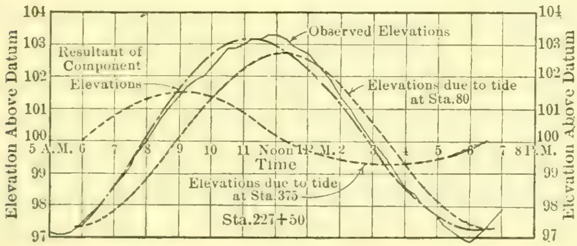

FIG. 37.

The discrepancy at the ends can be explained by the fact, as dealt with in detail in another part of this paper, that, at these stations, for outflow a smaller and for inflow a greater percentage of reduction should be applied to the float observations, in order to arrive at the true mean velocity of the cross-section. If the reduction factors of 68 and 85%, respectively, which are believed to be the true values for reducing the center float velocities for the terminus sections, had been applied to the observed velocities at Stations 80 and 375, the differences of the computed and the reduced velocities would be negligible at these sta-

tions also. If, furthermore, it is considered that the curves of observed velocities have been derived from float observations, which evidently are affected by accidental disturbances, such as the wind, the passing of larger boats, and local changes in the cross-section, it may be concluded that the computed results are well within 10% of the actual conditions.

It cannot be denied that the uncertainty attached to the selection of the friction coefficient cannot be entirely eliminated, and therefore one should not expect as close results as hydraulic engineers are accustomed to; for instance, in the case of masonry conduits or pipes in which the motion of the water is uniform. It would fairly seem, however, that a theory which for this exceedingly complex problem furnished results within 10% of actual observations, should be regarded as satisfactory and reliable.

By a further inspection of the velocity diagrams, it will also be noted that there is a marked difference in the character of the computed and observed curves, the latter being much steeper near slack water than the former. The observed curve has been reduced from the float observations to mean velocities by applying the factor, 0.78, to all observed center float velocities. The difference in the two curves would tend to show that this factor varies considerably with the velocity, and should be taken very much smaller at low velocities.

This proposition, which, so far as the writer knows, lacks authority, might be worthy of independent investigation and experimentation.

Finally, it will be seen that the duration of the westerly current is appreciably shorter than that of the easterly current, which fact is due directly to the low velocities generated by the Buzzards Bay tide during the second half of the cycle. It will also be noted that the computed time of maximum velocities and the computed time of slack water check within a few minutes with the actual occurrence of these extremes.

The equations submitted herewith are general, and their use is not restricted by any other consideration than a canal of reasonably uniform cross-section. They can also be used for predicting conditions in canals of varying cross-section, by dividing them into reaches, where the cross-sections can be considered as sensibly uniform, and by determining the constants, C, D, C', and D' in the General Equations (*11*) and (*12*) from the conditions that both ξ and h must be equal at the

limits of two consecutive reaches. This calculation, of course, will be very much more complicated than that shown in this paper, but it is the only way of getting reliable results in the case where the canal is long and the cross-section varies considerably.

Approximate Methods.

For a uniform canal connecting tidal bodies of water, approximate formulas to predict velocities may be used, with certain restrictions, giving results which for maximum conditions will not be very far from the actual velocities, and, at any rate, can be used for the evaluation of the friction coefficient to be used in the exact formulas.

9.—Uniform Flow Formulas.—The formulas used to determine the velocity of uniform flow in canals (with the exception of the exponential formulas, which have been disregarded in this discussion) all have the form of the well-known Chezy formula

$$v = C \sqrt{R\, S}$$

where

v denotes the uniform velocity of the flow;

R " " hydraulic radius of the section $= \dfrac{\text{wetted area}}{\text{wetted perimeter}}$;

S " " slope of the water surface or canal bed, supposed to be parallel to each other

$$= \frac{\text{difference in elevation at the ends}}{\text{length of canal}};$$

C is a coefficient, dependent chiefly on the roughness of the perimeter, also on the slope and the hydraulic radius of the cross-section.

The several formulas in use differ only in the different methods of establishing this coefficient.

In order to be able to apply these formulas to a canal in which tidal motion takes place, the fact must be disregarded that the bottom for most of the time has a slope different from that of the surface, and one must consider the surface slope, assumed to be a straight line, as being $= S$. An average value of R must be introduced into the formulas. These assumptions made, it is evident that the velocity becomes proportional to the square root of the difference in elevations

at the ends of the canal. In Fig. 38, therefore, are platted the mareographs at Stations 80 and 375 in such a way that the difference in levels can be read off for every time point. Being interested in the maximum value of the velocity, let the following formulas be applied to the maximum difference in levels, which occurred at 6.45 A. M., and amounted to 5.85 ft. The hydraulic radius of the canal at this time averages 18.6 ft., the stage of the water being very low.

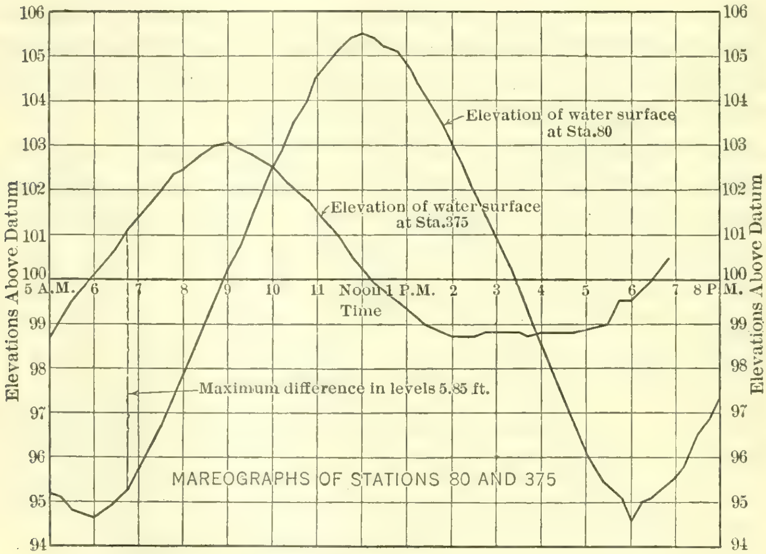

FIG. 38.

(*a*).—*Bazin's Formula.*—The great pioneer of hydraulic researchers, Bazin, has derived a formula, intended for the calculation of flow in open channels, in which it is assumed that the coefficient, C, does not vary sensibly with the slope. The formula (in foot units) reads

$$v = \frac{87}{0.552 + \frac{m}{\sqrt{R}}} \sqrt{R\,S} = C\sqrt{R\,S}$$

where the values of m, corresponding to the frictional constants given on page 1147, are 0.85, 1.30, and 1.75, respectively.

With this last value of m, C becomes equal to 90.6, and

$$v = 90.6\sqrt{18.6 \times \frac{5.85}{29\,500}} = 5.5 \text{ ft. per sec.}$$

(*b*).—*Eytelwein's Formula.**—This is written in a different form from the Chezy formula, expressing the velocity as a function of the head, that is, the difference in elevations at the ends of the canal.

$$v = \sqrt{\frac{2\,g}{1 + \frac{K\,L}{R}}}\,\sqrt{h_1 - h_2}.$$

Of course, there is no difficulty in bringing this formula back to the form of the Chezy formula, in which case it would read

$$v = \sqrt{\frac{2\,g\,L}{R\left(1 + \frac{K\,L}{R}\right)}}\,\sqrt{R\,S} = C\,\sqrt{R\,S}$$

where K is the frictional constant and g is the acceleration of gravity. With the value, $K = 0.00756$, and the other quantities substituted,

$$v = \sqrt{\frac{2 \times 32.2}{1 + \frac{0.00756 \times 29\,500}{18.6}}}\,\sqrt{5.85} = 5.38 \text{ ft. per sec.}$$

(*c*).—*Kutter-Ganguillet Formula.*—This formula assumes the coefficient, C, to vary with the slope as well as with the roughness of the bed and the hydraulic radius. The expression for the velocity in this formula (in foot units) reads,

$$v = \frac{41.6 + \frac{1.811}{n} + \frac{0.00281}{S}}{1 + \left(41.6 + \frac{0.00281}{S}\right)\frac{n}{\sqrt{R}}}\,\sqrt{R\,S} = C\,\sqrt{R\,S}.$$

The values of n, corresponding to the frictional constants, are 0.02, 0.025, and 0.03, respectively.

With this last value of n, C becomes equal to 83, and

$$v = 83 \times \sqrt{18.6 \times \frac{5.85}{29\,500}} = 5.04 \text{ ft. per sec.}$$

This result was used for evaluating the coefficient, f_0', in the harmonic solution of the problem.

(*d*).—*Remarks.*—The application of the foregoing formulas to the Cape Cod Canal and the comparison of the results with the observations, show a reasonable check, in so far as maximum velocities are concerned. This was to be expected in the case of a comparatively

* This formula was used by the officers of the U. S. Coast and Geodetic Survey, when asked by the writer to predetermine the velocities in the Cape Cod Canal.

short canal, where the retardation of the wave due to friction amounts only to a few minutes, and therefore the maximum slope of the stream is sensibly equal to the maximum difference in water levels at the ends divided by the length of the canal. In the case of long canals, the formulas for uniform flow become unreliable, giving much lower results than the actual. In the case of short canals, and considerable differences in head, on the other hand, the formulas will become inaccurate because the basic assumptions that the slope of the water and that of the canal bed are nearly parallel will not be true. It can be stated, on the basis of trial calculations not reproduced here, that, for a canal similar to the Cape Cod Canal in cross-section and roughness of bed, the uniform-flow formulas will furnish too high or too low velocities, according as the length is such as to make $q\ L$ smaller or greater than 0.45.

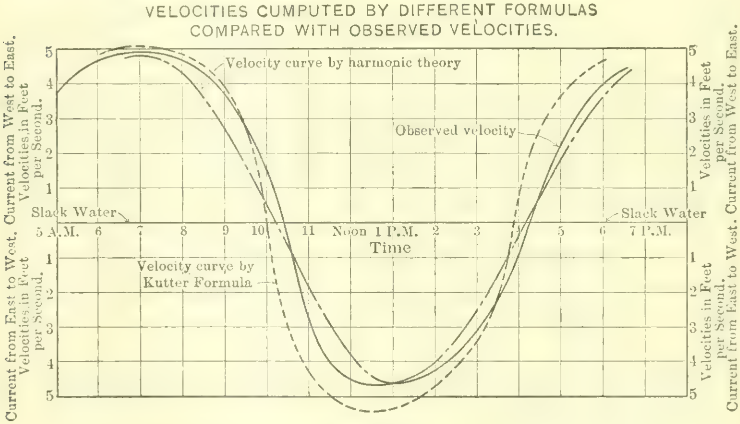

FIG. 39.

It can also be seen that this method can only be applied in cases where there is considerable difference in the amplitude of the tides at the two ends of the canal, or where there is a considerable difference in the phases of the tides. With equal tides and coincident phases, these formulas would furnish zero velocities for all sections, whereas, actually, there will be considerable motion at the ends of a canal several miles long, the velocities diminishing toward the middle of the canal. Fig. 39 shows the velocity in the Cape Cod Canal as computed by Kutter's formula for the whole range of observations on August 26th, 1915, as compared with the results by the modied Airy-Lévy formulas and the actual measurements.

(10).—Formula for Permanent Non-Uniform Flow.—Due to the fact that maximum velocity in the canal occurs near the time when the difference in elevations at the ends of the canal is a maximum, and due also to the fact that a change from this condition is at a comparatively slow rate, the canal for maximum velocity can be considered as one with very nearly horizontal bottom, connecting two seas of different elevations, discharging water from the higher to the lower.

The method of computation hereafter shown is given in "Handbuch der Ingenieurwissenschaften" by Professor Bubendey of Hamburg, but, to the writer's knowledge has never been published in English.

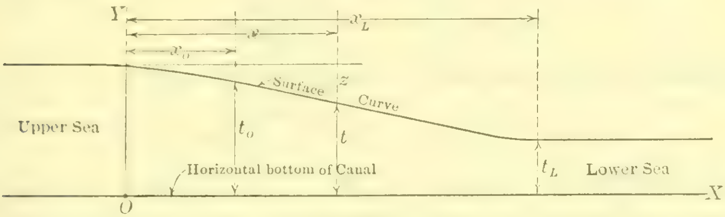

FIG. 40.

The differential equation of non-uniform flow in channels reads

$$\frac{d\,z}{d\,x} = -\frac{\alpha}{g}\,\frac{Q^2}{F^3}\,\frac{d\,F}{d\,x} + \frac{Q^2\,X}{F^3\,C^2},$$

where Q = constant discharge of canal;
F = area of cross-section;
X = wetted perimeter of cross-section;
g = acceleration of gravity;
C = the constant in Chezy's formula, $v = C\sqrt{R\,S}$.
α = a constant, the value of which is 1.11 according to St. Venant.

The meaning of the other symbols is shown in Fig. 40.

There will first be derived the working formula for a parabolic cross-section and then be shown how a trapezoidal section can be transformed into an equivalent parabolic section. As an allowed approximation, assume that the width of the water surface, $b = X$, the wetted perimeter. Then

$$X = b = 2\sqrt{P\,t}$$

where P = the parameter of the parabola and t = the maximum

depth of the cross-section. From the well-known properties of the parabola,

$$F = \frac{4}{3} \sqrt{Pt}\ t, \text{ and } F^3 = \frac{64\ P^{\frac{3}{2}}\ t^{\frac{9}{2}}}{27}$$

$$\frac{d\,F}{d\,x} = 2\,P^{\frac{1}{2}}\ t^{\frac{1}{2}}\ \frac{d\,t}{d\,x},\ \frac{1}{F^3}\ \frac{d\,F}{d\,x} = \frac{27}{32\,P\,t^4}\ \frac{d\,t}{d\,x},\ \frac{X}{F^3} = \frac{27}{32\,P\,t^4}$$

As $dz = -\,dt$, according to the figure, substituting the other values in the differential equation, we get

$$\frac{d\,t}{d\,x} = \frac{27\,\alpha\,Q^2}{32\,g\,P\,t^4}\ \frac{d\,t}{d\,x} - \frac{27\ Q^2}{32\,C^2\,P\,t^4}$$

and from this, separating the variables,

$$d\,x = \frac{\alpha\,C^2}{g}\ d\,t - \frac{32\ C^2\ P}{27\ Q^2}\ t^4\ d\,t$$

Integrating between the limits, x_0, t_0, and x_L, t_L, we get

$$x_L - x_0 = \frac{\alpha\,C^2}{g}\ (t_L - t_0) - \frac{32\,C^2\,P}{135\ Q^2}\ (t_L^{\,5} - t_0^{\,5})$$

which equation is a complete solution of the problem. With given water elevations at the ends of the canal, and given dimensions of the cross-section, the discharge, Q, can be found. After Q is computed, the values of x corresponding to different values of t (between t_0 and t_L) can be found and the surface curve platted. With the help of the surface curve, the areas of the wetted cross-sections at any point can be measured, and consequently the velocities can be determined by dividing Q by the corresponding areas.

Let F_1, b_1, and F_2, b_2, be the area and surface width of the trapezoidal cross-sections at the outlet and inlet ends of the canal, respectively, then the parameter of the equivalent parabola is

$$P = \frac{1}{2}\left(\frac{b_1^{\,3}}{6\,F_1} + \frac{b_2^{\,3}}{6\,F_2}\right).$$

Let $a_1{}'$ and $a_1{}''$ represent the distance of the vertex of the parabola from the lower sea level, at the outlet and inlet ends, respectively, and let $h =$ the difference in levels, then

$$\frac{2}{3}\,a_1{}'\ b_1 = F_1 \text{ and } \frac{2}{3}\,(a_1{}'' + h)\ b_2 = F_2$$

and the average

$$a_1 = \frac{a_1{}' + a_1{}''}{2}.$$

If t_L' and t_0' are the actual depths of the trapezoidal cross-sections at the outlet and inlet, then

$$t_L = t'_L + (a_1 - t'_L) = a_1$$

and

$$t_0 = t_0' + (a_1 - t_L')$$

t_0 and t_L being the maximum depths of the substituted parabolic sections at the inlet and outlet, respectively.

Applying the formula for the conditions prevailing at the time of the maximum difference in levels on August 26th, 1915, the water elevations at Stations 80 and 375 are found to be 95.25 and 101.1, respectively, the current being easterly. These elevations being platted in Fig. 41,* the following data are obtained:

$x_0 = 0 \qquad t_0' = 29.75$ ft.

$x_L = 29\,500 \qquad t_L' = 23.90$ ft. $\qquad h = 5.85$ ft.

$F_1 = 3\,532.4$ sq. ft. $\qquad b_1 = 195.6$ ft.

$F_2 = 4\,745.1$ sq. ft. $\qquad b_2 = 219.0$ ft.

$C = 83$ (by Kutter).

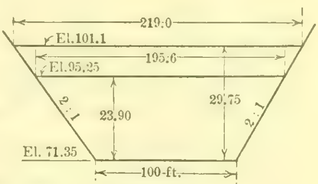

FIG. 41.

$$P = \frac{1}{2}\left(\frac{195.6^3}{6 \times 3\,532.4} + \frac{219.0^3}{6 \times 4\,745.1}\right) = 361.$$

$$a_1' = \frac{3 \times 3\,532.4}{2 \times 195.6} = 27.1$$

$$a_1'' = \frac{3\left(4\,745.1 - \frac{2}{3} \times 5.85 \times 219\right)}{2 \times 219} = 26.7$$

$$a_1 = \frac{27.1 + 26.7}{2} = 26.9$$

$$t_L = 26.9 \text{ ft.} \qquad t_0 = 29.75 + (26.9 - 23.9) = 32.75 \text{ ft.}$$

Substituting these values in the equation, we have

$$29\,500 = \frac{1.11 \times 83^2}{32.2}(26.9 - 32.75) - \frac{32 \times 83^2 \times 361}{135\, Q^2}(26.9^5 - 32.75^5)$$

$$= -1\,390 + \frac{588\,000}{Q^2}(32.75^5 - 26.9^5)$$

$$Q^2 = \frac{588\,000\,(32.75^5 - 26.9^5)}{30\,890} = 349\,000\,000$$

$$Q = 18\,700 \text{ cu. ft. per sec.}$$

* Fig. 41 is not drawn to scale.

The maximum velocity will be at the outlet, or at Station 80, and is given by

$$v_{max.} = \frac{18\,700}{3\,532.4} = 5.28 \text{ ft. per sec.}$$

This result shows that the method here presented gives maximum velocities, the magnitudes of which differ only slightly from those derived by the use of the uniform flow formulas. The remarks appended to the latter apply to this method without exception; in approximate calculations it should be used for short canals, to which the uniform flow formulas do not apply on account of the appreciable difference between the slope of the water and the slope of the canal bed.

11.—French Academy of Sciences' Formula.—In 1886, Count Ferdinand de Lesseps, Chief Engineer of the French Isthmian Canal Company, asked the Academy of Sciences to institute an investigation about the influence of tidal motion of the Pacific and Atlantic on the motion of the water in an open Panama canal. A committee, consisting of Messrs. Daubrée, Favé-Lalanne, de Jonquières, Boussinesq, and Bouquet de la Grye, was appointed by the Academy to answer the request of Count de Lesseps, and reported on the subject on May 31st, 1887. The report in its essence says:

1.—That the tidal variations at the Atlantic end of the proposed canal are so small as to be neglected.

2.—That experience shows that in a canal communicating on the one end with a sea of variable level, on the other end with a lake of constant level, the amplitude of the tidal curve diminishes uniformly from the sea to the lake, and further that the retardation of the tide is proportional to the distance, that, therefore:

If Y = half amplitude of the tide,
l = length of the canal,
w = velocity of propagation of the tides, and
t = time, in seconds, measured from the instant of low water at the sea, and 24 lunar hours being equal to 2 η, then the level, y, with respect to the mean canal or sea level, at a distance, x, from the sea, will be

$$y = -Y\left(1 - \frac{x}{l}\right) \cos. \left(2\,t - \frac{x}{w}\right).$$

3.—That, in accordance with what has been observed on similar canals, particularly on the Suez Canal between Suez and the Bitter

Lakes, the velocity of propagation of the tide can be represented by the formula:

$$w = \sqrt{g\left(H + \frac{3}{2}\,y\right)} \pm K\,v,$$

where H = depth of canal below mean sea level,

v = velocity of current,

K = constant (0.4 at flood tide, 1.2 at ebb tide).

4.—That, from the levels which have been derived by the foregoing equations for any moment and for two mutually not too distant places, the velocity of the current for any moment may be computed by applying the formula

$$v = C\,\sqrt{R\,S}.*$$

The velocities should be computed for, say, every half hour of the cycle, and the results tabulated; from this table the maximum velocity can easily be pointed out.

In applying this method of computation to a canal connected to tidal seas at both ends, the influences of the individual tides on the water elevations should be treated separately and the results added, with due regard to the sign of the slopes. The velocities should be calculated on the basis of the resulting slopes.

In order to check this formula against the Cape Cod observations, let it be applied to Station 227+50, admitting for the velocity of wave propagation the approximative value of

$$w = \sqrt{g\,H} = \sqrt{32.2 \times 28.65} = 30.7 \text{ ft. per sec.}$$

The maximum velocity at Station 227+50 occurred at 7.00 A. M., and low water at Station 80 was at 5.35 A. M., and at Station 375 at 2.50 A. M.

Taking first the influence of the tide at Station 80, and considering that the lunar time is 0.97 × solar time (nearly):

For x = 14 750 ft.

$$y = -5.4\left(1 - \frac{14\,750}{29\,500}\right)\cos.\frac{0.97\left(2 \times 85 \times 60 - \dfrac{14\,750}{30.7}\right)360^\circ}{86\,400}$$

$$= -5.4 \times 0.5 \times \cos.\,39^\circ\,20' = -2.09 \text{ ft.}$$

* As a matter of fact, the Committee, having in mind the special problem of the Panama Canal, suggested the formula, $v = 56.86\,\sqrt{R\,S} - 0.07$ (metric).

At a distance 1 000 ft. from this station, or for $x = 15\ 750$ ft.,

$$y = -5.4\left(1 - \frac{15\ 750}{29\ 500}\right)\cos.\frac{0.97\left(2 \times 85 \times 60 - \dfrac{15\ 750}{30.7}\right)360°}{86\ 400}$$

$$= -5.4 \times 0.466 \times \cos.\ 39°\ 10' = -1.95\text{ ft.}$$

The slope, S, $= \dfrac{2.09 - 1.95}{1\ 000} = 0.00014.$

The influence of the tide at Station 375 will, by inspection of Figs. 29 and 30, increase this slope, the half amplitude being 3.05 ft.

For $x = 13\ 750$ ft.

$$y = -3.05\left(1 - \frac{13\ 750}{29\ 500}\right)\cos.\frac{0.97\left(2 \times 250 \times 60 - \dfrac{13\ 750}{30.7}\right)360°}{86\ 400}$$

$$= -3.05 \times 0.534 \times \cos.\ 119°\ 30' = 0.802\text{ ft.}$$

For $x = 14\ 750$ ft.

$$y = -3.05\left(1 - \frac{14\ 750}{29\ 500}\right)\cos.\frac{0.97\left(2 \times 250 \times 60 - \dfrac{14\ 750}{30.7}\right)360°}{86\ 400}$$

$$= -3.05 \times 0.5 \times \cos.\ 119°\ 20' = 0.749\text{ ft.}$$

The slope, S, $= \dfrac{0.802 - 0.749}{1\ 000} = 0.000053.$

The resultant slope $= 0.00014 + 0.000053 = 0.000193.$

Substituting this in the Chezy formula, and applying Kutter's coefficient for C, we get

$$v = 83\sqrt{18.6 \times 0.000193} = 4.97\text{ ft. per sec.,}$$

a result which checks the observed velocities very closely.

The method here described has been thoroughly discussed and criticized in a paper by Dr. C. Lely,* who finds, in connection with studies relating to the Panama Canal, that the formula can be considered only as fairly approximate, but by no means accurate, because it does not comply with the law of continuity.

Judging from the remarkably close result furnished by the formula of the French Academy of Sciences when applied to the Cape Cod Canal, one must conclude that this formula gives the best approximate method for predicting the numerical value of tidal currents in canals

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* *Proceedings*, Amsterdam Academy of Sciences, April 26th, 1907.

connecting two tidal seas. The objections to it which can be raised, namely, that the assumption given under Paragraph 2 is not exact, because the amplitude of the tidal curve cannot be a linear function of the distance, and the method of computing the velocities by the application of a uniform flow formula, not being accurate, are not serious for maximum values. The inaccuracy of the formula is much more evident at low velocities. In other words, near the time of the reversal of the current, the velocities deduced by its use change direction with the reversal of the slope, though it is a fact that, for a considerable time after the slope inclination has changed, the current still flows in the original direction. For canals as long as to make $q\ L$ equal 0.55 or more, this formula should be used to evaluate the friction coefficient, f_0', to be used in the formulas derived by harmonic analysis.

Conclusions.

The writer has endeavored to give a complete treatise on the question of tidal currents in canals, so as to enable the reader to gain a clear idea of this complex phenomenon, and has indicated the methods which can be used in predicting the magnitudes of such currents for given conditions.

There is no doubt in the writer's mind that the Airy-Lévy formulas, derived by harmonic analysis, with the modification proposed by the writer, represent the only rational method for solving the hydrodynamic problems arising within a tidal canal. This method is fully treated under the heading "The Solution of the Problem by Harmonic Analysis." The formulas for the evaluation of the elevation of the water and the velocity of the current at any instant are given in Section 5, page 1141, the proposed modification of the same to meet actual conditions is discussed in Section 6, page 1146, and the numerical application of the method is shown in Section 7, page 1147.

It is questionable, however, whether or not, in practice, the refinements and complicated numerical work connected with the harmonic analysis are justifiable. The principal assumptions on which all the formulas are necessarily based, namely, the prevalence of a uniform cross-section and a uniform friction coefficient, are never quite true in the practical case of a canal dug in earth; and both cross-section and friction coefficient are apt to change materially with time. A number of other factors influencing the motion of the water, such

as wind, drainage of the water-shed tributary to the canal, etc., cannot be taken into consideration at all, and, *a priori,* reduce all analytical results to the grade of a more or less fair approximation. In addition, it should be noted that the designer of a canal is interested chiefly in the maximum velocity which may occur under given tidal conditions, and the behavior of the current for other values has little interest for him.

It would appear, therefore, that the approximate methods of calculation shown in this paper should be used for the practical evaluation of the velocities in canals subjected to tidal influences.

Three such methods have been suggested herein: In Section 9, pages 1158-59, Bazin's, Eytelwein's, and Kutter's formulas relating to uniform flow are given. The formulas based on permanent but non-uniform flow are treated in Section 10, page 1161, and those recommended by the French Academy of Sciences, *i. e.,* taking the velocity of propagation of the tidal wave into consideration, are shown in Section 11, page 1164.

In the detailed discussion of these different formulas, the writer has shown that none of them can be adopted for general application, but that each of these approximate methods is best suited for certain distinct classes of canals, and, therefore, they should be used with caution.

The numerical results obtained by applying the approximate methods to the Cape Cod Canal would indicate that the formulas for uniform flow will give good results for a canal having the same characteristic features, both as to design and physical composition of channel bed. The approximate frictional factor to be used for a canal of this type is 0.03 in Kutter's formula. For other types of canals no such comparative statement can be made, because of the small number of existing tidal canals and the lack of accurate information in regard to the velocities.

The writer believes, however, that the results obtained by the application of the rational method (which are as accurate as can be at the present status of the science) furnish excellent means to determine the limitation of the use of the three practical methods presented. Therefore, taking these results as a measure of accuracy, the writer makes the following recommendations:

(*a*).—For tidal canals, the length, cross-sectional dimensions, and frictional characteristics of which are such as to make $q\ L$ less than 0.35, use the formulas for permanent non-uniform flow.

(*b*).—For canals where $q\ L$ lies between 0.35 and 0.55, use the formulas for uniform flow.

(*c*).—For canals where $q\ L$ is greater than 0.55, use the formulas of the French Academy of Sciences.

In $q\ L$, L is the length of the canal, in feet, and q has the value given in Equation (*8*), representing the influences of the form and area of the cross-section and the frictional properties of the channel bed. This value, q, being dependent on a great number of variable factors, cannot be represented by a diagram of general applicability, and should be evaluated in each individual case. To overcome the ambiguity caused by the fact that q is also dependent on the velocity to a certain extent, as a rule, a few trial calculations, as explained on page 1146, will be necessary to arrive at the correct governing value of the product, $q\ L$.

Applying the proposed criteria to canals of the type of the Cape Cod Canal, *i. e.,* having nearly the same cross-section and the same consistency of channel bed, and being subjected to tidal differences of about the same magnitude, it can be stated that, for predicting the maximum velocities in such uniform canals:

If they are not longer than about 5 miles, use the formula for permanent non-uniform flow;

If their length is between 5 and 8½ miles, use the formulas for uniform flow (Bazin, Kutter, etc.);

If they are longer than 8½ miles, use the formulas of the French Academy of Sciences.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

PROGRESS REPORT OF THE SPECIAL COMMITTEE TO CODIFY PRESENT PRACTICE ON THE BEARING VALUE OF SOILS FOR FOUNDATIONS*

To the American Society of Civil Engineers:

The Special Committee appointed

(1) To codify present practice on the bearing value of soils for foundations, and

(2) To report on the physics of soils in relation to engineering structures,

respectfully submits this report of progress.

During the past year your Committee held three meetings; but its work was suspended during the greater part of the year, because of lack of necessary funds, and an investigation into the work of the Committee by the Board of Direction. Quite recently, the Board of Direction approved the continuation of the work of the Committee, and appropriated funds (believed to be sufficient) to complete the report on the Classification of Soils, and to standardize apparatus and procedure for defining soils.

Your Committee has noted the discussion on its last report in relation to the classification of soils; but the practical experience of the Committee during the past year indicates that the classification may be further simplified. Modifications have been proposed by the United States Bureau of Mines, and the United States Geological Survey, which will have the earnest consideration and study of your Committee.

Your Committee reiterates that it has been found impractical to codify the data collected on classes of soils, because of the impossibility of identifying the soils and interpreting values for bearing capacity. Your Committee, however, in resuming its activities, will endeavor to

* Presented to the Annual Meeting, January 17th, 1917.

present the data collected in useful form; and, also, will consider methods of making practical tests and necessary observations for collecting future data.

Your Committee finds that it is vitally important to obtain as a basis of its work as much scientific information as possible on the physical properties of soils; and that, in so far as it is practical, the study of the physics of soils should be conducted along the general lines in vogue in the study of materials of construction. With this in view your Committee has been co-operating with the Bureau of Standards, Dr. S. W. Stratton, Director. The cordial appreciation of the work of that Bureau is here acknowledged, and the scientific report of the Sub-Committee is submitted as Appendix A. All this experimental work is being carried on by the U. S. Bureau of Standards at no expense to the American Society of Civil Engineers, because the subject is recognized as being of National importance. During the year the attention of other scientific and technical institutions will be directed to the unusual opportunity presented for co-operation in this work.

On the request of your Committee, the Carnegie Library, of Pittsburgh, through its Technology Department, Mr. Harrison W. Craver, Librarian, and Mr. Ellwood H. McClelland, Technology Librarian, has compiled a "Bibliography on the Physical Properties and Bearing Capacity of Soils", at no expense to the American Society of Civil Engineers. This is presented as Appendix B. The cordial appreciation of your Committee is extended to the Carnegie Library, of Pittsburgh, for this comprehensive collection of references.

The references are classified as follows:

1. Natural Phenomena.
   - Erosion.
   - Sedimentation and Silting.
   - Slides, Slips, and Subsidences.
2. Chemical and Physical Properties of Soils.
   - Theory.
   - Testing.
     - Methods and Results.
     - Instruments.
   - Bearing Value.
   - General and Miscellaneous Properties.
3. Granular Materials.
   - Sand and Gravel.
   - Quicksand.
   - Grain.
   - Miscellaneous.
4. Foundations.

5. Retaining Walls, Including Lateral Earth Pressure.
6. Piles.
    General.
    Theory and Formulas.
    Tests.
    Pile-driving.

There are 859 references, each one of which has been actually examined, and all may be found in the Carnegie Library, of Pittsburgh. Titles of articles and names of journals are given in full. Dates, volume numbers, and inclusive paging are given. In most cases the nature of the article is indicated by a brief explanatory note, and all bibliographies accompanying books or magazine articles are mentioned.

During the past year your Committee has taken part in a conference with representatives of various scientific societies and Government Bureaus at the Bureau of Standards, Washington, D. C., which resulted in the adoption of a Standard Screen Scale for wire sieves, as detailed in the report of the Bureau of Standards, presented as Appendix C.

Respectfully submitted on behalf of the Committee.

ROBERT A. CUMMINGS,
Chairman.

JAN. 16TH, 1917.

COMMITTEE:

WALTER J. DOUGLAS, *Secretary,*
EDWIN DURYEA,
E. G. HAINES,
ALLEN HAZEN,
J. C. MEEM,
ROBERT A. CUMMINGS, *Chairman.*

APPENDIX A

PROGRESS REPORT OF SUB-COMMITTEE ON RESISTANCE OF SOILS.

Your Sub-Committee has been engaged on the second division of the investigational work outlined by your Committee in the previous report, *viz.*, the determination of the physical characteristics of soils by laboratory methods. The results of experimentation submitted at this time are to be considered tentative. Only typical data, for the purpose of illustration, will be presented, as much of the experimental work is incomplete, and the data are not fully organized at the present writing.

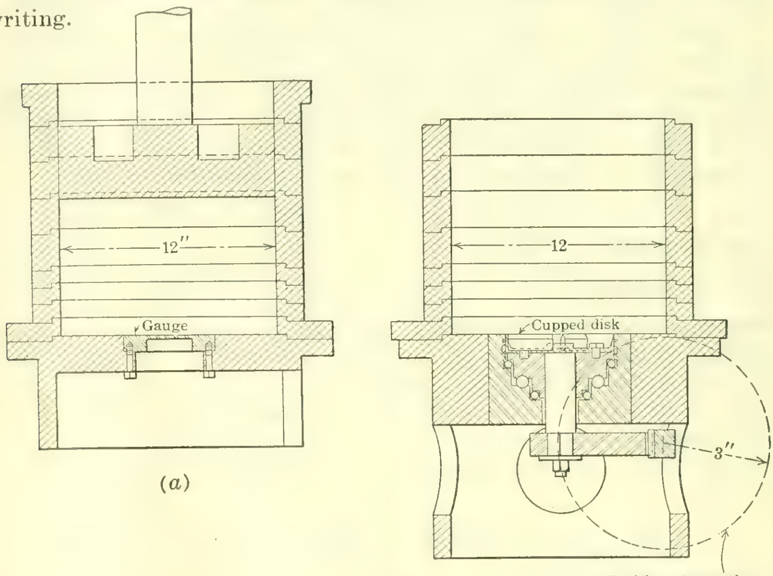


Fig. 1.

Standard Methods of Grading and Separation.—Your Sub-Committee has held frequent discussions with your Chairman relative to standard methods of grading and separation, but no recommendations can be made at this time. It is felt that an efficient method of grading and separation of granular materials is imperative, if an earth is tested under standard conditions.*

Standard for Laying of Earth.—Numerous tests have been made with the nested cylinder device described in a previous report.† (Fig. 1.) A material reduction in the range of variation of the data has

* The report of the conference on a standard screen scale for testing sieves has since been issued.

† *Proceedings*, Am. Soc. C. E., Papers and Discussions, March, 1916, p. 343.

been effected with cylinders of this type. The earth is deposited under approximately identical conditions in different tests, with uniform pressure of the piston, and without eccentricity from the axis. The apparatus is found to be convenient in determining densities of soil aggregates. A granular soil is "stroked off" at the desired height with a screed. A fine wire is used for clays.

General Plan of Tests.—Your Sub-Committee has stated that the resistance and other properties of earths are functions of a number of variables. The structural variables are defined by your classification. The property variables are defined by the mechanics of materials. A number of the variables are functionally related, *e. g.*, cohesion-water content, strain-density, friction-plasticity, etc. Your Sub-Committee has sought to determine the law of variation of particular functions in terms of the more important variables, when the others are held constant or may be practically ignored.

The general soil which is hypothecated in the planning of tests, in conformity with your classification, is as follows: Grains or particles, of different material, shape, and mean diameter, are conceived as surrounded by a matrix, wholly or partly filling the voids. Plasticity is the essential variable of this matrix, with water content a particular case. The particles are considered as incompressible under moderate pressures. They are slightly compressible and more or less disintegrable under heavy pressures. The matrix is subject to more or less compaction in the voids. A particular earth may be fairly approximated for laboratory study by a proper selection of the two components. A clay, for example, may be considered to be wholly matrix without grains, or finely comminuted grains without matrix; a sand may be considered as wholly grains without matrix, etc. The surfaces of grains are surfaces of limited stability, *i. e.,* slipping and readjustments occur at certain stages, and stress and strain functions and their derivatives are subject to finite discontinuities. The "film" or matrix surrounding particles is considered by soil analysts to be an essential factor in studying the stabilities and equilibriums which exist or are possible in different earths.

It has been found necessary, in investigational work of this nature, to duplicate experimental conditions almost identically in repeating tests. Otherwise, variations of several hundred per cent. between results will often occur, and it is difficult to draw conclusions from the tests in such cases. If uncertainties exist in the case of experiments conducted on a small scale under conditions of some refinement, they may be expected to exist *a fortiori* in the case of experiments on a larger scale under conditions approximating those of practice.

The granular earths are studied under a definite range of variables. The clays and mixed soils are treated independently, the plasticity of the latter being considered. Since the last report, experiments

have been confined to standard 20-30 Ottawa sand, the matrix considered being water, the quantities varying in 5% increments from zero moisture to saturated. Clays are being considered at the present time and out of the regular order, for the purpose of studying the workings of apparatus under extreme conditions.

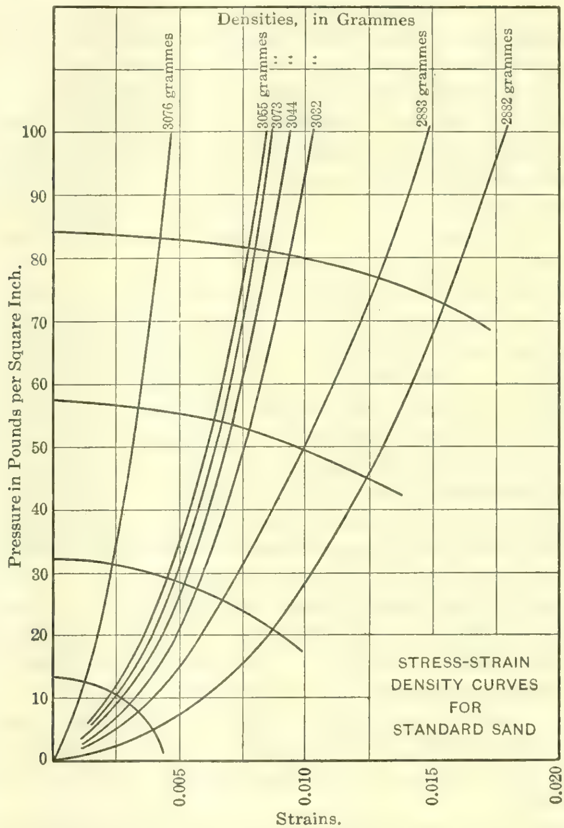


FIG. 2.

Remarks on Stress-Strain Relations.—Dry Sand.—Oven-dried sands have been tested in the cylinder, Fig. 1 (*a*). Loads were measured on the beam of a 200 000-lb. testing machine. Strains were measured by Ames' dials placed at opposite points on the piston. Curves of the type of Figs. 2 and 3 were found. For loads, $0 < p < 50$, stress is an increasing function of strain, showing that energy is mainly consumed in compacting voids. Stress is practically proportional to strain for $p > 50$, indicating partial equilibrium under internal frictions. Stress is a decreasing function of strain for most materials,

wherein flow and disintegration of particles as well as reduction of voids occurs, the effect of reducing voids apparently being to impose positive curvature and flow or breaking down of particles of negative curvature on the slope of curves, but this is not yet determined conclusively. The effect of repetitions of loads shows pronounced reduction in strain with mainly inelastic set, there being little resiliency in sand. In some cases twenty cycles were taken, the curves in Figs. 2 and 3 being reduced to the origin. There is a region of condensation at the 5th-20th interval, the curves approaching a limit or "frontier" without further appreciable set. The p axis (Fig. 4) is the stress-strain curve for the ideal case of incompressible particles with minimum voids.

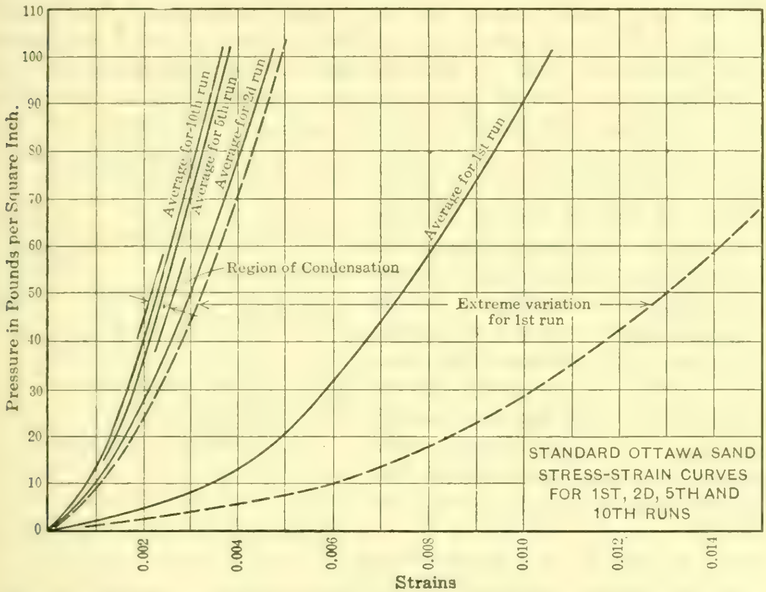


FIG. 3.

The s axis, on the other hand, is the curve for loosely laid earths.* The wide variations in strains during earlier runs indicate the need of a close determination of the density of earth, in the opinion of your Sub-Committee, if the results of different investigators are to be placed on an experimental parity.

Pressure, Strain, and Density Curves.—In practical experiments by engineers, the only reference to density, as a rule, is an approximate statement of the specific weight of the earth. Your Sub-Committee did not attach sufficient weight to this variable during former tests.

* When dry sand is dropped from a height into a box and "stroked off," a single tap will lower the surface as much as ¼ in.

Large variations in the physical properties accompany slight changes in density of the mass. The practical feasibility of determining the density in "prospecting" a soil will depend, evidently, to a large extent on the practical efficiency of the apparatus already devised by your Committee.

An attempt has been made to show, in the typical tests to be given, that for constant pressure the strain varies inversely with the density. The volume can be determined closely, as the nested cylinder is accurately machined. The exact weight of the earth is determined after each test. With a height of 5 in. under the piston, and weights measured to 1 gramme, the density is determined to within about 0.01 per cent. In the curves of Figs. 2 and 3 a height of sand in the cylinder of 1 in. was used, and equi-initial density curves are drawn for the first, fifth, and tenth runs, the load being removed in each individual case at 100 lb. per sq. in. and applied again at zero. The actual densities vary slightly with the pressure and strain, but this was ignored in drawing the present curves. The gradient $\left(\text{limit}\,\frac{\Delta D}{\Delta n}\right)$, or slope of steepest ascent, is measured along lines drawn normal to the contours, usually called "lines of flow." It is seen that a change of initial density of only 6% causes approximately a difference of from 300 to 400% in the strain on the initial "run up", the curves converging on the fifth and tenth runs, and the relative positions of the curves changing in some cases. On the first "run up" two curves are out of the natural order, which may denote an error in the measurements.

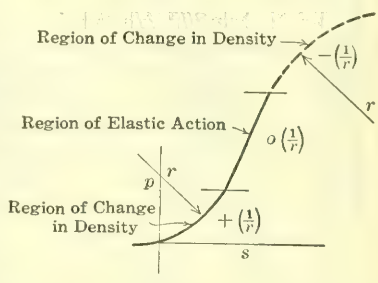


FIG. 4.

In the case of solids and fluids, the pressures, in general are proportional to the densities, as in the case of Boyle's law for gases. Granular and pulverulent media are stated by Boussinesq to possess properties intermediate to the former extremes. If this is the case, it should indicate that the density will play an important part in the standardization of tests for earths. A large number of tests will be needed to demonstrate the relative influence.

Gauges.—Your Sub-Committee has continued its experimental studies of the gauges described in the last report. The newer gauges are turned from solid pieces of tool steel, and are set flush with the walls of the containers, without reveals. Diaphragm portions are 2.526 in. in diameter to give 5 sq. in. area. The thicknesses of the diaphragm portions are 0.04, 0.08, and 0.12 in., or 1, 2, and 3 mm., respectively.

The gauges are screwed tight to the walls, without possibility of relative motion. The calibrations with mercury tubes and Ashcroft gauges show that the gauges are very dependable under hydrostatic pressures.

The hydrodynamic analogue of earth under pressure in a cylinder is the whirling fluid under action of gravity and centrifugal force. (Fig. 5). The lines of stress are theoretically nearly normal to the piston, and diverge to the walls and the base of the cylinder. Accordingly, there is theoretically less pressure on the diaphragm at the base of the cylinder than the mean applied stress on the piston. The tests with the 1-mm. gauge, on the contrary, have shown excess pressures on the diaphragm amounting to as much as 25% of the corresponding hydrostatic pressure, when the piston is within 1 in. of the diaphragm. The excess is reduced as this distance is increased.

Tests with the 3-mm. diaphragm on 1 and 5 in. of sand, show pressures greater than hydrostatic for 1 in. and less than hydrostatic for 5 in. of sand, respectively, (Fig. 6), the difference in ordinates, in the latter case, apparently representing the loss due to friction on the walls, which increases with the pressure and depth of the material. A study of a larger number of tests not here recorded indicates that the pressure depends not only on the thickness of the diaphragms, but also, to some extent, on whether the load is applied at the center or circumference of the rim of the piston, which is quite rigid.

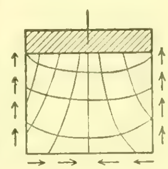


FIG. 5.

William Cain, M. Am. Soc. C. E., has pointed out to the Committee that the phenomenon of excess pressures occurred in the Illinois Experiment Station and State College tests, and to a greater degree than is here shown. This was attributed to lack of uniformity in the pressure. The matter will require futher investigation before definite knowledge, in this respect, can be obtained.

Conjugate Pressures.—Standard Sand.—Dry.—Conjugate pressures in a medium are pressures so related that either acts on a plane parallel to the direction of the other. (Fig. 7.) Principal stresses are special cases when the conjugate planes are perpendicular. The apparatus used by your Sub-Committee for determining the ratio of the conjugate pressures is shown in Fig. 1. The nested cylinder was used as the container. The density was measured as previously described. An accurately machined cap is placed on top of the cylinder. The piston, *a*, and the gauge, *b*, each of 5 sq. in. cross-section, are placed eccentrically 2 in. from the axis of the cylinder. The pressure is applied at *a*, and the uplift is measured at *b*.

According to Rankine's theory, the relation between the "active pressure" or effort at *a* and the induced pressure at *b* is $p_b = p_a \left(\frac{1 - \sin. \phi}{1 + \sin. \phi}\right)^2$, where p_a is the pressure on the piston, p_b is the pressure

on the gauge, and ϕ is the angle of internal friction for the soil. In an exact sense, the equation defines the mode of equilibrium at a definite point in a cohesionless material when the particles exert their maximum frictional resistances. Since it is impossible to devise mechanical means for determining the exact pressures and their laws of variation, p_a, p_b, and ϕ, as determined by the tests, are considered to represent mean values for the region affected. The angle, ϕ, is taken as a convenient parameter in making a comparative study of experimental results for different earths.

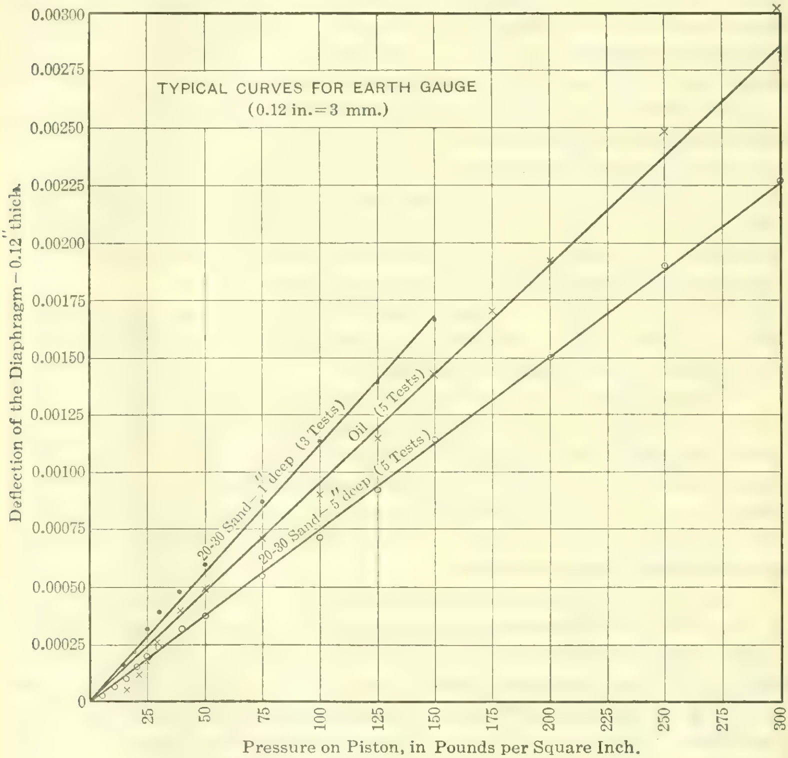


FIG. 6.

Piston pressures are applied and readings are taken on the beam of the 200 000-lb. testing machine. Gauge pressures in the case of the thin diaphragms are measured with an accurately calibrated Ames' dial, reading direct to 0.0001 and by estimation to 0.00001 in., and for thick gauges and greater refinement by the Bauschinger type of apparatus reading deflections to 0.000001 in. when the cathetometer is placed at 100 in. (Fig. 7).

There are shown in Fig. 8 a typical set of nine tests on standard, oven-dried sand. The gauge pressures, p_b, are platted as ordinates, the piston pressures, p_a, as abscissas. The ratio, $\frac{p_b}{p_a}$, from a particular curve, represents the value, $\left(\frac{1 - \sin.\ \phi}{1 + \sin.\ \phi}\right)^2$, corresponding to the particular pressure, p_a. The square root of this value gives the ratio of the applied vertical pressure to the induced lateral pressure. Theoretically, the curves should pass through the origin. Actually, there is a slight readjustment of grains with an increase in density under initial loads for values near the origin. The cohesion factor, k, in the equation, $q = p_n \tan.\ \phi + k$, has not been considered, but is discussed later.

From these curves there are deduced the mean values of the coefficient and angle of friction, as follows:

$$\begin{array}{l} \phi_1 = 39^\circ\ 20' : \tan.\ \phi_1 = f_1 = 0.82 \\ \phi_2 = 49^\circ\ 20' : \tan.\ \phi_2 = f_2 = 1.15 \end{array} \left\{ \begin{array}{r} 0 < p_1 < 100 \text{ lb. per sq. in.} \\ 100 < p_2 < 200 \text{ lb. per sq. in.} \\ \text{Density nearly constant.} \end{array} \right.$$

These results are somewhat higher than the values recorded in engineering pocketbooks, $\phi = 20$ to 35°. The percentage of variation from the mean is pronounced at the origin and beyond 120 lb. per sq. in.; the mean variation beyond 100 lb. per sq. in. varies approximately from $\pm$ 5 to $\pm$ 15 per cent. This variation can doubtless be reduced as additional study is given to the data, and the methods of tests are perfected.

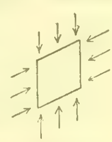


FIG. 7.

The induced pressure on the gauge increases quite uniformly with the applied pressure in the interval, $0 < p_1 < 100$. In the interval, $100 < p_2 < 200$, there appears to be equilibrium of the particles against motion on account of the internal friction. The gauge stood nearly stationary at times in this interval. In the interval, $200 < p_3 < 300$, the frictions appear to have been partly overcome with re-gearings of the grains taking place, probably on account of slight breaking down and slips at the peripheries of the grains. The tests submitted confirm to some extent the statement of G. H. Darwin, that internal friction is a function of the pressure and density.*

Coefficient of Friction Determined with Cup Disks.—Cohesion Factor, k, Neglected.—A number of tests have been made with the rotary cup-shaped disks (Fig. 1 (b)), and also with smooth steel disks. In the typical results to be quoted, cohesion has been ignored as quite negligible for dry sand. It will be considered with different percentages of moisture, and as methods become perfected.

* *Minutes of Proceedings*, Inst. C. E., Vol. LXXI (1883), pp. 374 *et seq.*

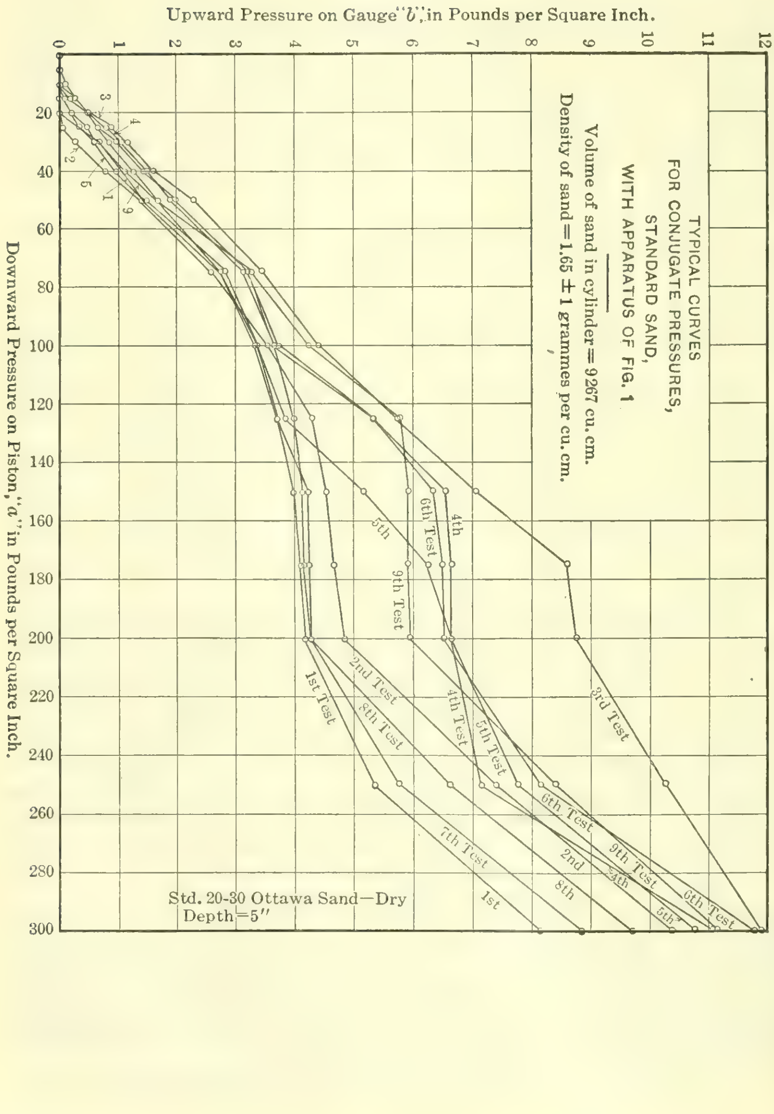

Upward Pressure on Gauge "b", in Pounds per Square Inch.
Downward Pressure on Piston, "a" in Pounds per Square Inch.
TYPICAL CURVES
FOR CONJUGATE PRESSURES,
STANDARD SAND,
WITH APPARATUS OF FIG. 1
Volume of sand in cylinder = 9267 cu. cm.
Density of sand = 1.65 ± 1 grammes per cu. cm.
Std. 20-30 Ottawa Sand—Dry
Depth = 5″
1st Test
2nd Test
3rd Test
4th Test
5th Test
6th Test
7th Test
8th Test
9th Test

FIG. 8.

In the working of this friction apparatus, it is found that two classes of coefficients can be determined, one for friction at impending motion, and another for friction of actual finite rotary displacements. In the case of incipient motion, the observations and pressures are recorded for actual motions of from 0.0001 to 0.0002 in. at the rack, and the data are calculated (Fig. 9). In the second case, finite motions of increments from 0.01 to 0.10 in. were taken at the maximum pressures recorded with the gauges. These limiting cases give frictions at impending motion and for small actual motions. Almost any pressure and coefficient of friction can be found between these limits, depending on the conditions of equilibrium for intermediate values.

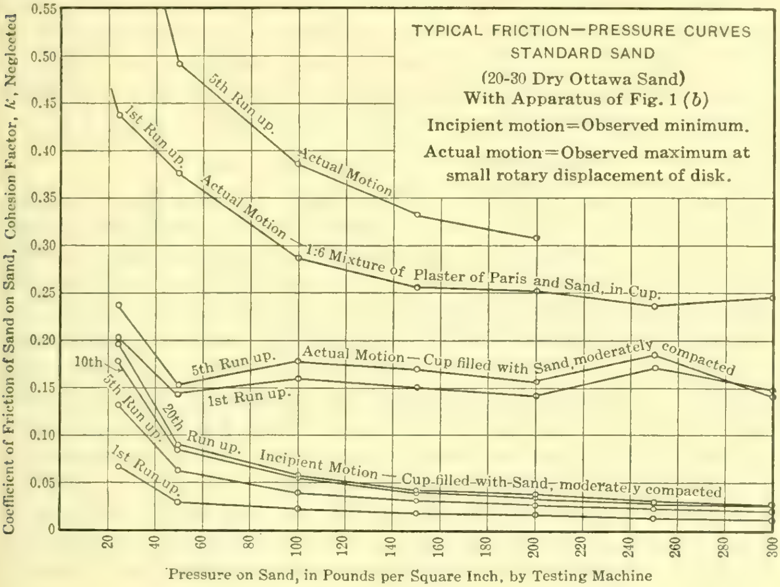


FIG. 9.

It is shown by the curves of Fig. 9 that sand in the cup-shaped disk does not exert the maximum friction. G. H. Darwin claimed* that the friction is a function of the pressure and the density of the mass, not only at any instant, but as well of that at some previous configuration or state. There is every indication that this will be so when the disks (or the double box arrangement) are used with granular materials. There is a partial confirmation of the theory of Boussinesq† that when the sand is held fixed at the boundary, slipping occurs on a near film of material, this friction at the film of slip representing the

* *Minutes of Proceedings*, Inst. C. E., Vol. LXXI, (1883), pp. 374 *et seq.*

† "History of the Elasticity and Strength of Materials," Vol. II, Part II, Article on Boussinesq by Karl Pearson.

maximum friction of sand on sand. To show this the sand was "restrained" in the cup disk, by mixing one part of sand with six parts of plaster of Paris, the mixture being richer on the surface. It is assumed that the grains at the surface of the disk are fixed, and that sliding occurs on the film just outside this surface. The coefficient of friction is about twice that for "free" sand in the cups.

A few preliminary trials with a knurled steel disk, the knurls being slightly larger than the grains, lead the experimenters to believe that this will be a more satisfactory form of disk for developing the maximum coefficient of a dry sand than the cup disks for comparative purposes in standard tests. In a material such as clay, however, which is not subject to much change in density after initial impaction, it is probable that the cup disks will be preferable. The cup disks will be more satisfactory in getting the cohesion factor, k, according to Cain's method,* since, with the knurled head, it will be quite impossible to compute the depressed areas or "pits" very closely.

Experiments with the Steel Disk.

Standard Sand.—Different Percentages of Moisture—Cohesion Factor, k, Neglected.—The law of variation of the coefficient of friction was studied for standard sand with different percentages by weight of water matrix added. The quantities of water varied from zero for dry sand to about 20% of the weight of the sand. The pressures are given in 50-lb. increments from zero to 300 lb. per sq. in. A smooth steel disk having the surface flush with the base of the cylinder was used in the tests. The coefficient of friction was determined under the conditions of a definite rotational displacement of the disk at the maximum pressures on the actuating piston of the lateral pressure cylinder, Fig. 1, as recorded by a mercury tube and pressure gauge. The pressure gauge was calibrated with the Emery tester. The frictional factor of the apparatus was determined by measuring the force necessary to turn the disk in oil, subject to pressure in the earth cylinder, and found to be small, varying from zero to 0.4 lb. per sq. in., when the oil pressure varied from no load to 200 lb. per sq. in. The earth was placed at a height of 5 in. in the cylinder, the different percentages of water being added by weight, and intimately mixed in the sand by rotating the mass.

In Figs. 10, 11, and 12, the calculated coefficients of friction are given as the ordinates, the percentages of water are given as the abscissas of the curves at 50 lb. sq. in. increments of load on the piston of the containing cylinder. The upper curve in all cases is the friction, calculated on the basis of average pressure recorded on the beam of the testing machine. The lower curve represents the corrected curve for the coefficient, f, on the basis of the pressure at the surface of the disk.

* "Cohesion in Earth: The Need for Comprehensive Experimentation to Determine the Coefficients of Cohesion." *Transactions*, Am. Soc. C. E., Vol. LXXX (1916), p. 1315.

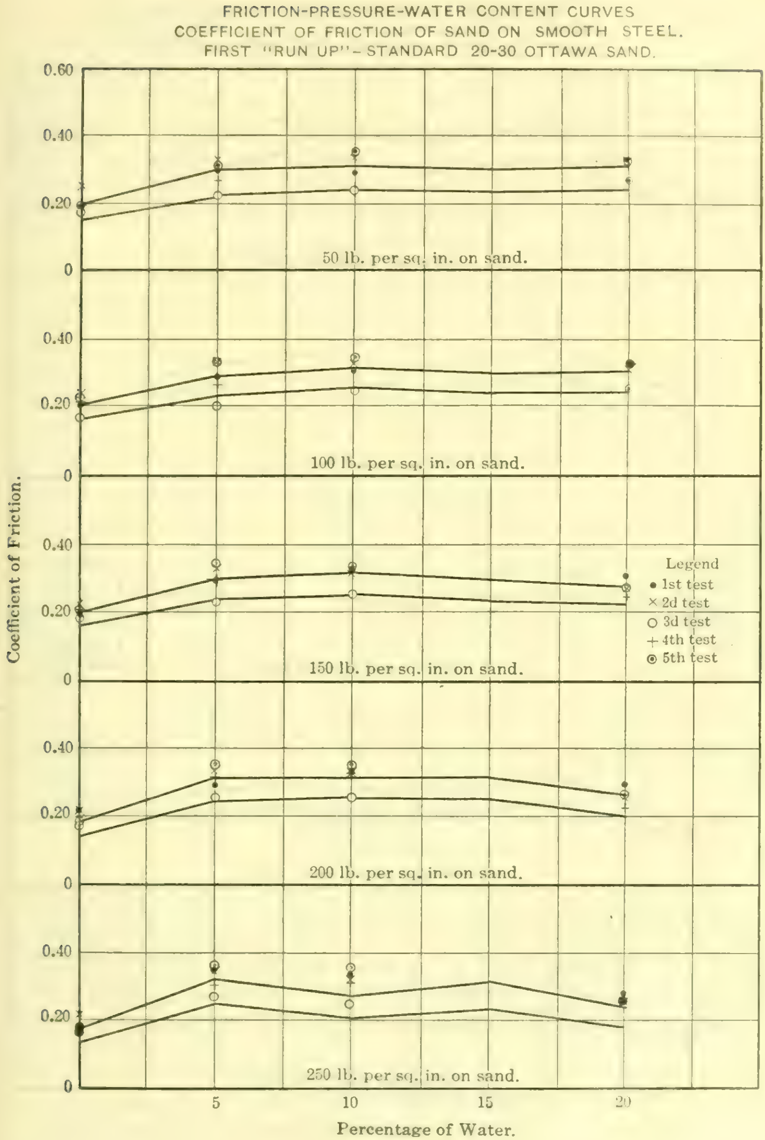

FRICTION-PRESSURE-WATER CONTENT CURVES
COEFFICIENT OF FRICTION OF SAND ON SMOOTH STEEL.
FIRST "RUN UP"- STANDARD 20-30 OTTAWA SAND.
Coefficient of Friction.
0.60
0.40
0.20
0
50 lb. per sq. in. on sand.
100 lb. per sq. in. on sand.
150 lb. per sq. in. on sand.
200 lb. per sq. in. on sand.
250 lb. per sq. in. on sand.
Legend
1st test
2d test
3d test
4th test
5th test
5
10
15
20
Percentage of Water.

FIG. 10.

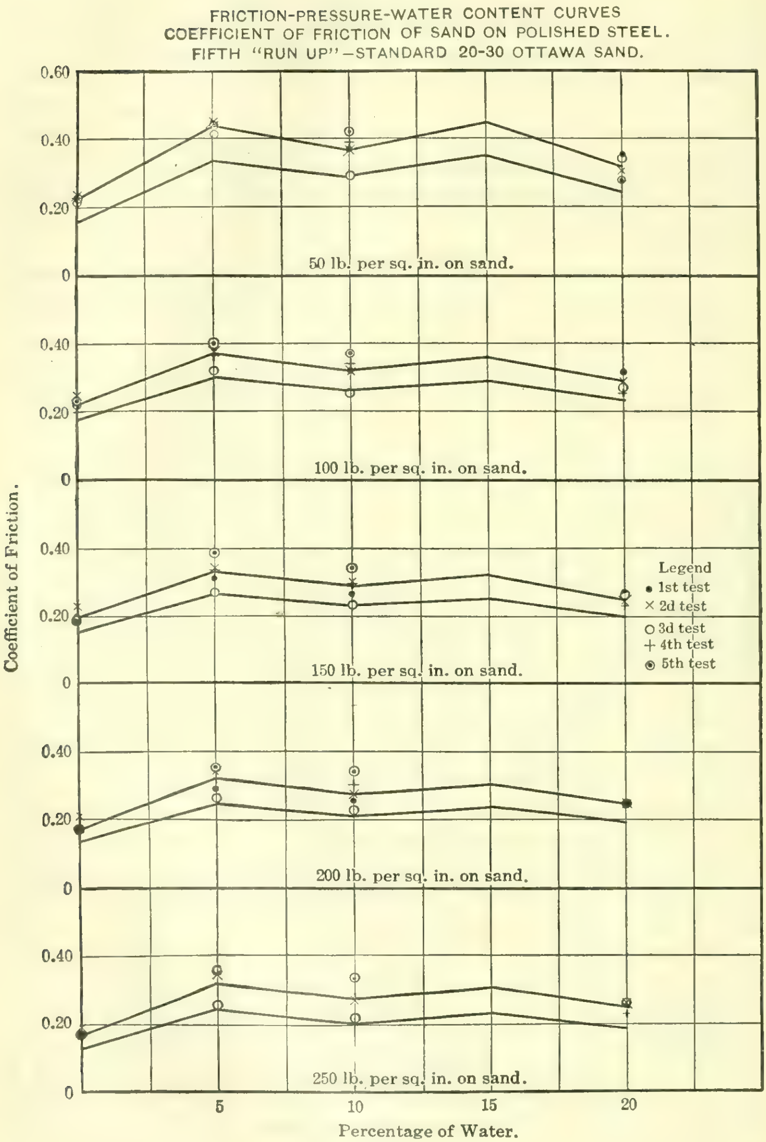

FRICTION-PRESSURE-WATER CONTENT CURVES
COEFFICIENT OF FRICTION OF SAND ON POLISHED STEEL.
FIFTH "RUN UP"—STANDARD 20-30 OTTAWA SAND.
0.60
0.40
0.20
0
50 lb. per sq. in. on sand.
100 lb. per sq. in. on sand.
150 lb. per sq. in. on sand.
200 lb. per sq. in. on sand.
250 lb. per sq. in. on sand.
Coefficient of Friction.
Legend
1st test
2d test
3d test
4th test
5th test
5
10
15
20
Percentage of Water.

FIG. 11.

This was determined by independent tests with the 3-mm. gauge, when it was placed at the bottom of the cylinder in the same position as the disk. The curves in each case are the means of five sets of tests.

It is seen that, for the first run, the coefficient of friction for steel on sand is a minimum for dry sand and a maximum for sand with 10% moisture, the slopes of the curves being quite uniform after 5% moisture is reached. The lower curves are closely parallel to the upper ones throughout. There is a "cusp" or re-entrant break in the curves for the fifth run in all cases, which cannot be explained, but is probably dependent on instrumental conditions. The sand at the fifth

FRICTION-PRESSURE-WATER CONTENT CURVES
COEFFICIENT OF FRICTION OF SAND ON POLISHED STEEL
Standard 20-30 Ottawa Sand

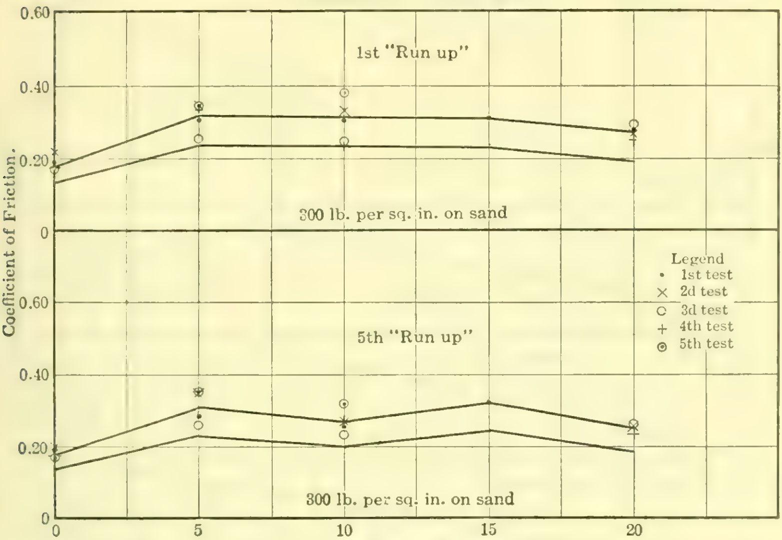


FIG. 12.

"run up" is believed to be in the more stable position corresponding to natural earths *in situ* than it is at the first "run up." The density was not determined in these tests, as its importance was not realized at the time. The results of tests for the first, second, and fifth runs indicate that the density affects the coefficient, but not to any such extent as it does the strain.

TESTS WITH SMOOTH DISKS.

Determinations of the Angle of Friction, ϕ, the Coefficient of Friction, f, and the Cohesion Factor, k, for Standard Sands Having Different Quantities of Moisture.—Cain has modified the Rankine equations, according to the method of Coulomb, to include the effect

of cohesion in a rational manner.* Determinations of the angle of friction, ϕ, the coefficient of friction, f, and the cohesion factor, k, have been obtained from the data in accordance with the method proposed by him.

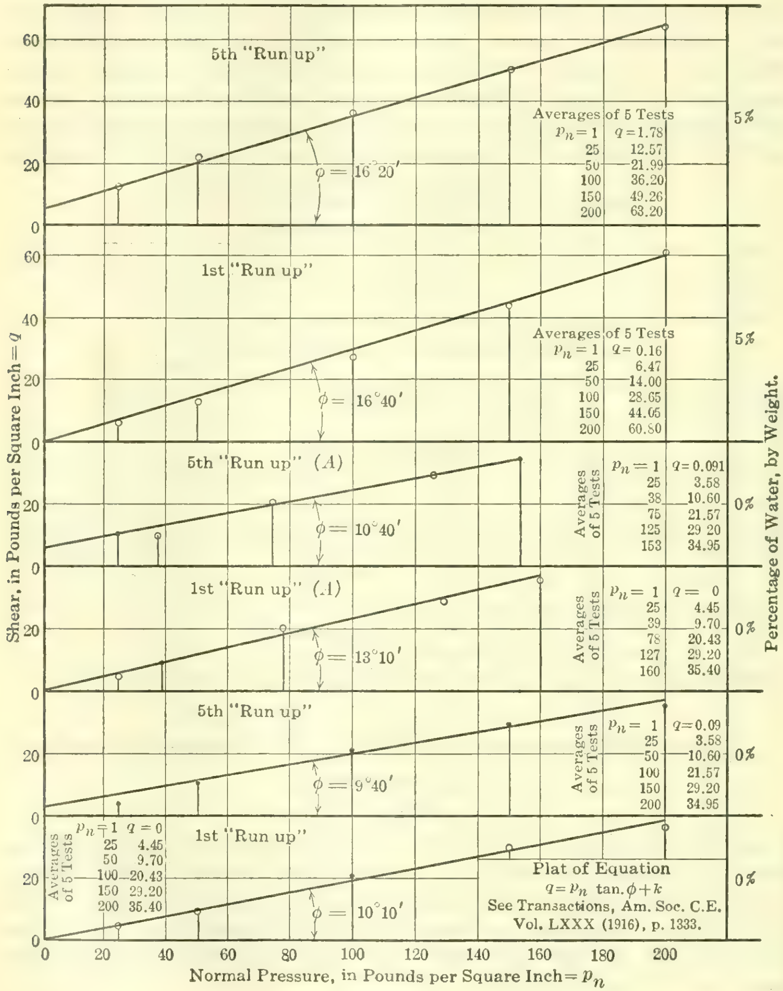


FIG. 13.

Standard sands having different percentages of moisture, as previously described, were placed in the cylinder, Fig. 1 (b), to a depth of 5 in. and subjected to a pressure, p_n, from the testing machine.

* "Cohesion in Earth, etc.", *Transactions*, Am. Soc. C. E., Vol. LXXX (1916), p. 1315.

The pressure required to move the disk against the pressure, p_n, on the piston was measured on the gauges attached to the 6-in. cylinder placed on the side of the main cylinder, as shown diagrammatically in Fig. 1 (*b*). The shear, q, on the face of the disk was calculated. The

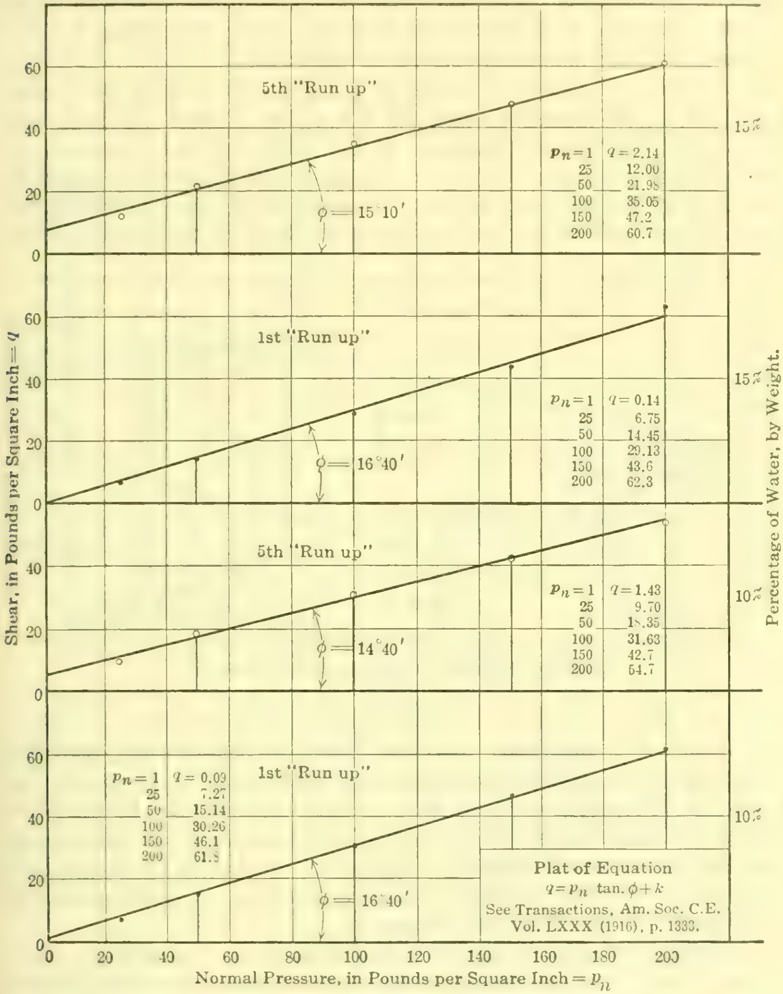


FIG. 14.

relation between q, p_n, ϕ, and k, according to Cain, is $q = p_n \tan. \phi + k$. Graphical representations of this equation for standard sands with different percentages of moisture are shown in Figs. 13, 14, and 15. The observed data, p_n and q, from which the plats were made,

are given for the different percentages of moisture, each value recorded being the mean of five tests. The shears, q, are platted as ordinates, the normal pressures as abscissas. The slopes of the straight lines passing through the platted observations represent the coefficient of friction, $f = \tan. \phi$. The angles between the different lines and the p_n axis represent the different values of the angle of friction, ϕ. The intercept on the q axis is the cohesion factor, k.

In the list of observed values of q and p_n given with the curves, the shear, q, has been corrected for the initial friction of the instrument from hydrostatic pressure alone. The normal pressure, p_n, represents the mean pressure on the large piston, as determined by the beam of the testing machine. The values, p_n, are doubtless influenced to some extent by the friction on the walls of the earth cylinder,

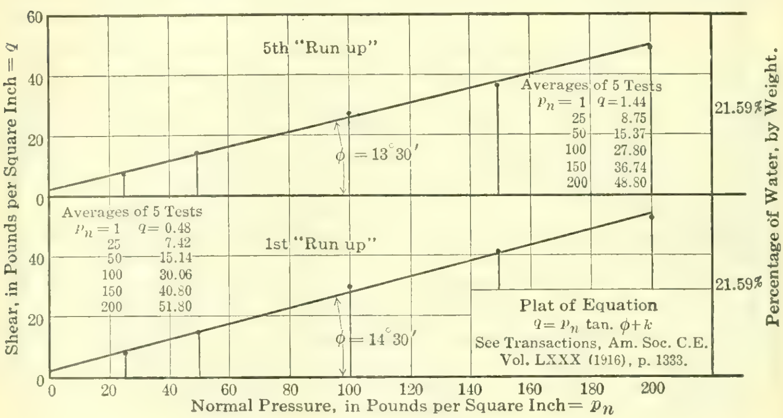
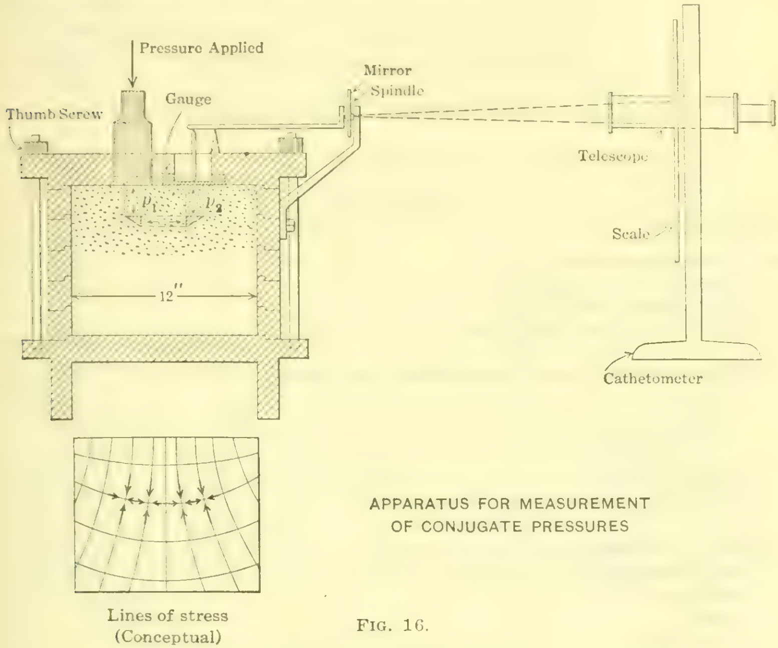


FIG. 15.

and cannot be considered absolute values of the normal pressures on the disk. In certain tests marked A the normal pressure recorded is that at the surface of the disk, as near as this can be predicted from independent tests. This pressure was determined on the assumption that the normal pressure recorded by the gauge in the case of sand is identical in amount with a hydrostatic pressure producing the same deflection of the diaphragm.

General Remarks.—Your Sub-Committee, in conclusion, again wishes to call attention to the provisional character of all the experimental data which have thus far been determined. The experimenters, up to the present time, have not been able to determine definitely the law of variation of the wall friction, nor to obtain an adequate control of this friction, although the problem is being carefully studied experimentally. The coefficients of a particular sand have

been shown to depend on the density. The few experiments which have been made on clay show that the viscosity is an important factor. The foregoing variables are believed to be among the important ones to be considered in standard tests of earths.



APPARATUS FOR MEASUREMENT
OF CONJUGATE PRESSURES

FIG. 16.

Acknowledgments.—Your Sub-Committee acknowledges the conscientious work of Messrs. E. Skillman and B. Hathcock in making the tests and calculations.

Committee of Bureau of Standards,

A. V. BLEININGER,
G. R. OLSHAUSEN,
J. H. GRIFFITH (*Chairman*).

APPENDIX B

BIBLIOGRAPHY OF PHYSICAL PROPERTIES AND BEARING VALUE OF SOILS.

(Compiled by the Carnegie Library of Pittsburgh)

CONTENTS

ABBREVIATIONS.

comp.	compiler	n. s.	new series
diag.	diagrams	p.	page or pages
dr.	drawings	pl.	plates
ed.	edition	pt.	part
ed.	editor	ser.	series
ill.	illustrations	v.	volume

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DU BOYS, P. Le Rhône et les Rivières a Lit Affouillable. Etude du Régime du Rhône et de l'Action Exercée par les Eaux sur un Lit a Fond de Graviers Indéfiniment Affouillable. 1 pl. 1879. (*Annales des Ponts et Chaussées*, ser. 5, v. 18, p. 141-195.)

DURAND-CLAYE, ALFRED. De l'Entrainement et de Transport par les Eaux Courantes des Vases, Sables, et Graviers. 6 diag. 1885. (*Annales des Ponts et Chaussées*, ser. 6, v. 10, p. 1165-1178.) Analysis of L. L. Vauthier's work on the subject.

DURHAM, WILLIAM. Suspension of Clay in Water. 1874. (*Chemical News*, v. 30, p. 57.) Abstract of a paper read before the Royal Physical Society of Edinburgh.

EADS, JAMES B. Hydrology of the Mississippi; a Reply to Gen. Abbot. 1879. (*Van Nostrand's Engineering Magazine*, v. 21, p. 154-158.) *See also* Abbot, v. 20, p. 1-6; Eads, v. 20, p. 211-229.

EADS, JAMES B. Hydrology of the Mississippi River; Review of Report by Humphreys and Abbot. 1878. (*Van Nostrand's Engineering Magazine*, v. 19, p. 211-229.) Severely criticizes report of Humphreys and Abbot, particularly the claim that the quantity of earthy matter held in suspension was not dependent in any way on the velocity of the current. *See* reply by Abbot (v. 20, p. 1-6); *also* Eads (v. 21, p. 154-158).

FARGUE. Etude sur la Largeur du Lit Moyen de la Garonne. 1882. (*Annales des Ponts et Chaussées*, ser. 6, v. 4, p. 301-328.)

FARGUE. Expériences Relatives a l'Action de l'Eau Courante sur un Fond de Sable. 1894. (*Annales des Ponts et Chaussées*,, ser. 7, v. 7, p. 426-466.)

FLAMANT. Compte Rendu des Expériences Hydrauliques Faites à Roorkee. 1882. (*Annales des Ponts et Chaussées*, ser. 6, v. 4, p. 43-96.) "Sediment," p. 82-83. Critical review of Major Cunningham's experiments on the Ganges Canal, India.

FORSHEY, C. G. Physics of the Lower Mississippi River. 1877. (*Proceedings*, Am. Assoc. for the Advancement of Science, v. 26, p. 148-173.) Discusses quantity and quality of sedimentary matter carried by stream, formation of bars, erosion of river banks, etc.

FRANCIS, JAMES B. On the Cause of the Maximum Velocity of Water Flowing in Open Channels being Below the Surface. 1878. (*Transactions*, Am. Soc. C. E., v. 7, p. 109-113, 168.) Seeks to explain the suspension of sediment in streams by the presence of a continuous upward flow of the water at the bed.

FREE, E. E. Phenomena of Flocculation and Deflocculation. 3 diag., 4 ill. 1910. (*Journal*, Franklin Inst., v. 169, p. 421-438; v. 170, p. 46-57.) "Bibliography," p. 54-57. Outlines the present knowledge of the phenomena of flocculation of particles suspended in water into aggregates. Discusses effect of various external conditions, mainly the presence or absence of soluble salts, acids, alkalies, and organic colloids. An appendix discusses the phenomena of flocculation in soils.

GILBERT, G. K. Transportation of Débris by Running Water. 263 p. ill. 1914. (United States Geological Survey, *Professional Paper 86*.) Based on experiments at Berkeley, Cal., with the assistance of Edward Charles Murphy. Finer débris is borne in suspension and quantitative measurement is not difficult. Primary purpose of this investigation was to determine the laws controlling the movement of coarser material swept along the channel bed, especially the relation of the load to the stream's slope and discharge, and to the degree of comminution of the débris. Velocities at stream bed were not satisfactorily measured. Experiments were made with both straight and crooked channels, and with both natural beds and flumes.

GILBERT, G. K. Transportation of Detritus by Yuba River. 1908. (*Bulletin*, Geological Society of America, v. 18, p. 657-659.)

GILBERT, G. K. United States Geological Survey's Hydraulic Laboratory at Berkeley, Cal. 1908. (*Science*, v. 50, n. s., v. 27, p. 469.) Outlines briefly experimentation to determine quantitatively the laws controlling the transportation of detritus by running water.

GRAHAM, JAMES C. On a Peculiar Method of Sand-Transportation by Rivers. 1890. (*American Journal of Science*, v. 140, p. 476.) Discusses an instance of the "transportation of siliceous sand upon the surface of the water, due to capillary floating." "Coarse sand can be floated away by a current of far less velocity than 0.45 miles per hour. . . . Indicates a possible explanation of the coarser particles of sand occasionally found in otherwise very fine deposits."

GRAS, SCIPION. Etudes sur les Torrents des Alpes. 1857. (*Annales des Ponts et Chaussées*, ser. 3, v. 14, p. 1-96.) Brings out the idea of saturation with solid material. Defines saturation in a stream as that state at which the least addition to the solid material already carried will cause a deposit; and carry-

ing power ("entrainement") as the total weight of material which a given stream in a state of saturation can carry. He assumes this power to vary directly with the velocity, density, and depth of the water, and, these quantities remaining constant, to vary with the volume, density, and form of solids. On these principles he explains erosion as a necessary consequence, when the saturation corresponding to the actual velocity is incomplete.

GUERARD, ADOLPHE. Mouth of the River Rhône. 3 diag., 1 pl. 1885. (*Minutes of Proceedings*, Inst. C. E., v. 82, p. 305-336.) Presents data on silt transportation. Believes that most of the solid matter discharged by the Rhône is pushed along its bed.

HAZEN, ALLEN. On Sedimentation. 2 diag. 1904. (*Transactions*, Am. Soc. C. E., v. 53, p. 45-88.) With discussion. Reviews Seddon's work on the subject, and discusses a number of assumptions for a theory of the process of sedimentation.

HILL, LOUIS C. Solving the Silt Problem. 1914. (*Engineering Record*, v. 70, p. 609-610.) Reservoirs suggested as a remedy for muddy streams used for irrigation purposes, where the silt would otherwise tend to clog the irrigation canals.

HOOKER, ELON HUNTINGTON. Suspension of Solids in Flowing Water. 1896. (*Transactions*, Am. Soc. C. E., v. 36, p. 239-324.) Discussion and correspondence, p. 325-339. Attributes suspension of sediment in flowing water to three causes: (1) resultant upward thrust due to eddies, caused by the irregular profile of the earth beneath, (2) resultant upward motion of solids due to fact that an immersed body tends to move faster than the mean velocity of the displaced water and in such motion tends to follow the line of least resistance, (3) viscosity of the water. Theories and process of suspension of solids are studied. Gives considerable attention to historical side of subject.

HUMPHREYS, A. A. Improvement of the Entrance to the Mississippi River by Jetties. (Annual Report [U. S.], Chief of Engineers, 1875, pt. 1, p. 959-964.) Discusses formation of bars by deposition of sediment where water from river meets salt water.

HUMPHREYS, A. A. Report of the Chief of Engineers [on the Ship Canal from the Mississippi River, near Fort St. Philip, to Isle au Breton Pass]. (Annual Report [U. S.], Chief of Engineers, 1874, pt. 1, p. 854-867.) Discusses sediment in rivers, and its transportation and deposition.

HUNT, T. STERRY. Deposition of Clays. 1874. (*Proceedings*, Boston Society of Natural History, v. 16, p. 302-304.) Discusses the conditions under which clay in suspension will settle.

JASMUND, R. Die Regulirung der Rhône. 5 diag., 3 pl. 1900. (*Zeitschrift für Bauwesen*, v. 50, p. 249-290.) Gives the quantities of silt and débris carried by the river.

JOHNSON, J. B. Results of Sand Wave and Sediment Observations. (Annual Report [U. S.], Chief of Engineers, 1879, pt. 3, p. 1963-1970.) Daily sediment observations were taken, and calculations made therefrom.

JOHNSON, J. B. Three Problems in River Physics. 1884. (*Proceedings*, Am. Assoc. for the Advancement of Science, v. 33, p. 276-288.) First of three problems discussed is "The Transportation of Sediment and the Formation and Removal of Sand-bars." Considers means of transportation of sediment, (1) by continuous suspension, (2) by discontinuous suspension, (3) by rolling along the bottom.

JOLLOIS. Démonstration Expérimentale de la Puissance de Suspension de l'Eau en Mouvement. 1893. (*Le Génie Civil*, v. 22, p. 278-279.)

——Condensed translation. 1893. (*Engineering News*, v. 29, p. 283.)

LEACH, SMITH S. Report . . . on Observations at Carrollton, La., December, 1879, to October, 1880. (Annual Report [U. S.], Chief of Engineers, 1883, pt. 3, p. 2209-2225.) Includes velocity measurements, and sediment observations.

LECHALAS. Note sur les Rivières à Fond du Sable. 1871. (*Annales des Ponts et Chaussées*, ser. 5, v. 1, p. 381-431.) Valuable paper, abstracted by "Gilbert" and by "Hooker". Takes exception to the theory attributing suspension to the phenomenon of relative velocities. He urges that this assumes flow in parallel filaments which corresponds in no wise to movements under great velocities. His explanation attributes suspension to repeated shocks from the molecules of water moving more rapidly than the suspended body, and to the action of eddies caused by the banks and bottom. Attempts to derive numerical results for the values of the mean depth, mean velocity, and fall, in alluvial rivers, which will follow the contraction in width throughout a given length by training walls.

LEINER. Zur Erforschung der Geschiebe- und Sinkstoffbewegungen. 9 diag., 4 dr. 1912. (*Zeitschrift für Bauwesen*, v. 62, p. 489-516.) Describes apparatus and methods of measuring silt movements.

McMATH, ROBERT E. Mississippi as a Silt Bearer. 1879. (*Van Nostrand's Engineering Magazine*, v. 20, p. 218-234.) Detailed discussion of observed facts and measurements.

McMATH, ROBERT E. Silt Movement by the Mississippi; Its Volume, Cause, and Conditions. 1882. (*Journal*, Assoc. Eng. Soc., v. 1, p. 266-275.) Considers volume of solid matter borne by river, mode of its conveyance, and conditions of deposit and removal.

——1883. (*Van Nostrand's Engineering Magazine*, v. 28, p. 32-39.)

McMATH, ROBERT E. Theory and Application of the Permeable System of Works for the Improvement of Silt-bearing Rivers. 1879. (*Engineering News*, v. 6, p. 353-355.) Gives views of writer, in form of skeleton argument, to show that in silt-bearing streams "moving material may be arrested and controlled as to location and forms of deposit by artificial means, and rendered permanent in such location and form."

MOLLER, M. Wasserklärung durch Absetzen. 5 diag., 4 dr. 1890. (*Schilling's Journal für Gasbeleuchtung und Verwandte Beleuchtungsarten sowie für Wasserversorgung*, v. 33, p. 8-13, 30-33.) Seddon's theory of sedimentation.

MOLLER, M. Zum Studium des Flussbaues. Die Stosscraft des Wassers, die Festigkeit der Sohle, das Gelfälle, das Geschiebe und die Bewegung feinerer Sinkstoffe. 2 diag. 1890. (*Zeitschrift für Bauwesen*, v. 40, p. 481-504.)

NAGLE, J. C. Progress Report on Silt Measurements. 1902. (United States Department of Agriculture. Experiment Stations, *Bulletin 104*, p. 293-324.) Irrigation investigations for 1900. Reports results of observations on silt conditions, principally on the Brazos and Wichita Rivers, Texas. Includes test methods and test data, and suggests remedies for silt problem.

NAGLE, J. C. Second Progress Report on Silt Measurements. 1902. (United States Department of Agriculture. Experiment Stations, *Bulletin 119*, p. 365-392.) Irrigation investigations for 1901. Reports results of observations on silt conditions, principally on the Brazos and Wichita Rivers, Texas. Includes test methods and test data, and suggests remedies for silt problem.

NAGLE, J. C. Third Progress Report on Discharge and Silt Measurements on Texas Streams. 1903. (United States Department of Agriculture. Experiment Stations, *Bulletin 133*, p. 196-217.) Irrigation investigations for 1902. Reports results of observations on silt conditions in several Texas streams. Includes test methods and test data, and suggests remedies for silt problem.

PARTIOT, HENRI LEON. Estuaries. 1 pl. 1894. (*Minutes of Proceedings*, Inst. C. E., v. 118, p. 47-77.) Abridged translation from the French.

PIERCE, RAYMOND C. Measurement of Silt-laden Streams. 1916. (United State Geological Survey. *Water Supply Paper 400-C*, p. 39-51.) Briefly describes the San Juan River and the gauging station established by the United States Geological Survey about 100 miles above the mouth of the river. Describes the methods used in overcoming the difficulties encountered in making discharge measurements of a stream having high velocities, large quantities of drift, shifting channel, and rapid fluctuations in its stage. Gives results in form of tables and curves.

[REPORT OF] MISSISSIPPI RIVER COMMISSION. 1883. (Annual Report, [U. S.] Chief of Engineers, pt. 3, p. 2111-2375.) Extended observations on sediment movement and sand waves.

REPORT OF THE MISSOURI RIVER COMMISSION. 1887. (Annual Report, [U. S.] Chief of Engineers, 1887, pt. 4, p. 2913-3132.) Numerous illustrations. Extensive observations on sediment movement and sand waves.

RICHARDS, ROBERT H., and WOODWARD, A. E. Velocities of Bodies of Different Specific Gravity Falling in Water. 1890. (*Transactions*, Am. Inst. Min. Engrs., v. 18, p. 644-648.) Tabulates different substances with regard to specific gravity and to fall per second in water.

RIEDEL, JOSEF. Ueber Geschiebführung und Murgänge der Wildbäche nebst ihrer Bedeutung für die Arlbahn. 9 diag. 1871. (*Zeitschrift*, Oesterreichischen Ingenieur-und Architekten-Vereines, v. 23, p. 113-117, 151-154.) Examples of dangerous "murgänge", canals through which semi-fluid mass passes at considerable velocity, but with little deposit.

SCHEERER, THEODOR. Einige Beobachtungen über das Absetzen aufgeschlemmter pulverförmiger Körper in Flüssigkeiten. 1851. (*Annalen der Physik und Chemie*, v. 170, p. 419-429.)

SCHLEPPKRAFTGESETZ. 1905. (*Zeitschrift*, Osterreichischen Ingenieur-und Architekten-Vereines, v. 57, pt. 1, p. 46, 169-170.) For water, with regard to transportation of silt and débris. Letters to editor by F. Kreuter and H. Engels.

SCHULZE, FRANZ. Die Sedimentär-erscheinungen und ihr Zusammenhang mit verwandten physikalischen Verhältnissen. 1866. (*Annalen der Physik und Chemie*, v. 217, p. 366-383.)

SEDDON, JAMES A. Clearing Water by Settlement; Observations and Theory. 3 diag., 1 dr., 7 pl. 1889. (*Journal,* Assoc. Eng. Soc., v. 8, p. 477-492.)

SEDDON, JAMES A. Notes on Sediment Observations of 1879 at Saint Charles, Missouri. (Annual Report, [U. S.] Chief of Engineers, 1887, pt. 4, p. 3090-3096.)

SIEDEK, RICHARD. Studie über die Bestimmung der Normalprofile geschiebeführender Gewässer. 9 diag., 4 pl. 1905. (*Zeitschrift,* Oesterreichischen Ingenieur- und Architekten-Vereines, v. 57, pt. 1, p. 61-73, 77-84.) Data for many rivers.

SILT AND SCOUR. 1 diag. 1906. (*The Engineer,* London, v. 102, p. 391-392.) Review of Bellasis's theory of erosion and silt formation advanced in his book "Hydraulics with...Tables".

——Abstract. 1907. (*Le Génie Civil,* v. 50, p. 275.)

SUTER, CHARLES R. Report on Portion of the Third Subdivision of the Mississippi Route. (Annual Report, [U. S.] Chief of Engineers, 1875, pt. 2, p. 496-521.) Discusses (p. 502-504) movement of sand suspended in stream.

THOMAS, B. F., and WATT, D. A. Improvement of Rivers. ed. 2. 2 v. 1913. Wiley. Includes considerable material on transportation of sediment, erosion and protection of banks, etc.

THOULET, J. Dosage des Sédiments Fins en Suspension dans les Eaux Naturelles. 1889. (*Comptes Rendus Hebdomadaires* des Séances de l'Académie des Sciences, v. 109, p. 831-832.)

THOULET, J. Expériences Relatives à la Vitesse des Courants d'Eau ou d'Air, Susceptibles de Maintenir en Suspension des Grains Minéraux. 1884. (*Annales des Mines, Mémoires,* v. 164, p. 507-530.)

THOULET, J. Expériences Relatives à la Vitesse des Courants d'Eau ou d'Air, Susceptibles de Maintenir en Suspension des Grains Minéraux. 1885. (*Annales des Ponts et Chaussées,* ser. 6, v. 9, p. 492-500.) Abstract of a paper in *Bulletin* de la Société Minéralogique de France. Experiments to determine the force required to keep particles of different sizes and densities suspended in water.

THOULET, J. Expériences sur la Sédimentation. 1890. (*Comptes Rendus Hebdomadaires* des Séances de l'Académie des Sciences, v. 111, p. 619-620.)

THOULET, J. Recherches Expérimentales sur la Vitesse des Courants d'Eau ou d'Air Susceptibles de Maintenir en Suspension des Grains Minéraux. 1883. (*Comptes Rendus Hebdomadaires* des Séances de l'Académie des Sciences, v. 97, p. 1513-1514.)

ULLER, K. Ueber den Verdrängungswiderstand fester Körper in Gasen und Flüssigkeiten. 1907. (*Annalen der Physik,* v. 340, p. 179-196.) Mathematical paper devoted to mechanics of suspension.

VAUTHIER, L. L. L'Entraînement et le Transport par les Eaux Courantes des Vases, Sables et Graviers. (*Mémoires et Compte Rendu des Travaux* de la Société des Ingénieurs Civils, 1885, pt. 2, p. 29-36.) Discusses action of flowing water, by which materials heavier than water are transported or held in suspension.

——Condensed translation. 1884. (*Engineering News,* v. 12, p. 211.)

WELCH, ASHBEL. [Presidential] Address. 1882. (*Transactions,* Am. Soc. C. E., v. 11, p. 153-180.) Discusses (p. 160-165) transportation of silt by Mississippi River and erosion of river banks.

WHEELER, WILLIAM HENRY. Application of the Transporting Power of Water to the Deepening and Improvement of Rivers. 2 diag. 1889-1890. (*The Engineer,* London, v. 68, p. 343-344, 383; v. 70, p. 42-43.) Attempts to show that the transporting power of water may be economically applied to the improvement of rivers by breaking up shoals, or the natural bed of rivers, by mechanical means and by mixing the material with the water and allowing it to be carried away to the sea or estuary in suspension.

WHEELER, WILLIAM HENRY. Tidal Rivers; Their Hydraulics, Improvement, Navigation. 467 p. 1893. Longmans, London. "Transporting Power of Water," p. 59-69.

WILLIS, BAILEY. Conditions of Sedimentary Deposition. 1893. (*Journal of Geology,* v. 1, p. 476-520.) Studies for students. Discusses erosion, transportation, distribution, chemical deposition, etc.

SLIDES, SLIPS AND SUBSIDENCES.

ANDREWS, HORACE. Earth Settlement in City Streets. 1906. (*Municipal Engineering,* v. 31, p. 361-366.) Paper before American Society of Municipal Improvements. Largely a compilation of material previously published on the properties and behavior of clayey soils.

BARROWMAN, JAMES. Slips in a Sandbank. 1 ill. 1902. (*Transactions*, Inst. Min. Engrs., v. 23, p. 154.) Notes on photograph of section in a British sandbank, clearly showing lines of slip.

BAUMGARTEN, KARL. Thunder Mountain Landslide. 1910. (*Mining and Scientific Press*, v. 101, p. 698-699.) Describes progress of landslide or mud flow.

BLACK, R. P. Remedies for Landslides and Slips on the Kanawha and Michigan Railway. 3 dr., 1 pl. 1911. (*Transactions*, Am. Soc. C. E., v. 71, p. 1-10.) With discussion.

BOLTE. Die Rutschungen an der Bebra-Hanauer Eisenbahn. 7 pl. 1871. (*Zeitschrift für Bauwesen*, v. 21, p. 69-82, 251-267, 379-390.)

BORRIES, von. Erhöhung des Bahndammes zwischen Hamburg und Bergedorf, unter besonderer Berücksichtigung der aufgetretenen Rutschungen. 1 dr., 1 pl. 1891. (*Zeitschrift für Bauwesen*, v. 41, p. 525-532.)

CAMBIE, H. J. Unrecorded Property of Clay. 1902. (*Transactions*, Can. Soc. C. E., v. 16, p. 197-199.) Discussion, p. 200-215. On landslides and their prevention.

——1907. (Am. Ry. Eng. and Maintenance of Way Assoc., *Bulletin No. 88*, p. 22-34.)

——Abstract. 1903. (*Engineering News*, v. 49, p. 38.) *See also* letter to editor, p. 104.

CANADA—GEOLOGICAL SURVEY. Report on the Great Landslide at Frank [Alberta]. 17 p., 14 ill., 2 charts, 2 maps. 1903. (Extracts from pt. 8 of the Annual Report of the Department of the Interior of Canada, 1903.)

CARPENTER, FRANK G. Creep in the Panama Canal; Conditions Caused by, and the Method of Dealing with, the Large Masses of Moving Earth and Rock. 1912. (*Mines and Minerals*, v. 33, p. 39-41.)

CARTAULT, Note sur les Glissements de Terrains dans les Tranchées Argileuses de la Ligne de Paris a Lyon entre Brunoy et Bois-le-Rois. 1894. (*Annales des Ponts et Chaussées*, ser. 7, v. 8, p. 377-392.) Discusses treatment of land slides in clayey trenches.

CARTER, HENRY H. Settlement of the Embankment between Squantum and Moon Island, Boston Main Drainage Works. 1892. (*Journal*, Assoc. Eng. Soc., v. 11, p. 355-362.) Gives results of observations on settlement of mud embankment, overlaid by gravel and filling material.

CHARIE-MARSAIENES. Mémoire sur les Travaux du Bief de Partage du Canal du Nivernais. 8 pl. 1848. (*Annales des Ponts et Chaussées*, ser. 2, v. 15, p. 1-112.) Presents data on earth pressures in connection with the building of the canal, describes slides and methods of their prevention, p. 21, 23, 31, 35, 69, 103, 105.

CLARKE, D. D. A Phenomenal Land Slide. 1904. (*Transactions*, Am. Soc., C. E., v. 53, p. 322-397.) Discussion, p. 398-412. Describes surveys and explorations, made over a long period, on slopes of two reservoirs of the city of Portland, Ore., to determine dimensions of landslide, its cause and means for prevention.

COMBER, W. G. Dredging Work on the Panama Canal Slides. 1915. (*Engineering News*, v. 73, p. 753-757.)

COMOY. Notice sur Divers Travaux de Consolidation de Terrains Eboulés. 3 pl. 1875. (*Annales des Pont et Chaussées*, ser. 5, v. 10, p. 8-51.) An account of various works undertaken for repairing landslides.

——Abstract. 1875. (*Minutes of Proceedings*, Inst. C. E., v. 42, pp. 268-271.)

COOPER, ROBERT ELLIOTT. Causes of Earth Slips in the Slopes of Cuttings and Embankments of Railways, and How to Prevent or Remedy Them. 1899. (*Minutes of Proceedings*, Inst. C. E., v. 138, p. 383-385.) States that "slips in embankments are chiefly of two descriptions. First, where the material composing the embankment slips; secondly, where the surface of the ground upon which the embankment rests slips away on the underlying strata. Gives means of remedying."

——1899. (*The Engineer*, London, v. 87, p. 612.)

——1899. (*Engineering*, v. 67, p. 826.)

CORNISH, VAUGHAN. On Landslides Accompanied by Upheaval in the Culebra Cut of the Panama Canal. 1913. (*Engineering*, v. 96, p. 443-444.) Author's view is that the mistake in not foreseeing the slides was due to disregard of chemical considerations, of the chemical action of rain water on the previously protected fragments ejected from volcanoes. The frequent occurrence of large upheavals was due to stratification.

——Abstract. 1913. (Report of the Eighty-third Meeting, British Assoc. for the Advancement of Science, p. 609.)

CORNISH, VAUGHAN. Panama Canal and the Philosophy of Landslides. 1913. (*Edinburgh Review,* v. 217, p. 21-42.) Deals mainly with description and occurrence of landslides along the Panama Canal.

COST OF SLIDES AND BREAKS IN CULEBRA CUT, PANAMA CANAL, IN CUBIC Yards of Excavation. 1912. (*Engineering News,* v. 68, p. 607.)

CURTIS, W. G. Notes on a Mountain Slide. 1 diag., 1 dr., 3 ill. 1891. (*Transactions,* Am. Soc. C. E., v. 24, p. 556-563.) With discussion. In Northern California.

DARTON, N. H. Novel Plan for Stopping a Landslide at Mount Vernon. 1915. (*Engineering News,* v. 73, p. 369-370.) The danger was averted by draining the water from a sandstone substratum. A masonry wall was built along the river's edge to prevent further undercutting by the waves.

DAWLEY, W. M. Drainage of Soft Spots in Old Roadbed. 1907. (*Proceedings,* Eighth Annual Convention, Am. Ry. Eng. and Maintenance of Way Assoc., v. 8, p. 541-554.) Drainage to prevent settlement of track at soft spots. Discusses classification of soft spots and slips.

——1907. (Am. Ry. Eng. and Maintenance of Way Assoc., *Bulletin No. 87,* p. 4-17.)

DAWSON, GEORGE M. Remarkable Landslip in Portneuf County, Quebec. 1 diag., 4 ill. 1899. (*Bulletin,* Am. Geol. Soc., v. 10, p. 484-490.)

DISASTER AT FRANK, N. W. T. An Account of the Slide on Turtle Mountain which Destroyed the Town of Frank and a Section of the Canadian Pacific R. R. 1903. (*Mines and Minerals,* v. 23, p. 559-560.)

DRAINING AND STEADYING SLIPS. 1904. (*Railroad Gazette,* v. 37, p. 317-318.)

DUMAS, A. Les Eboulements de la Tranchée Centrale du Canal de Panama; Etat Général d'Avancement des Travaux du Canal. 1913. (*Le Génie Civil,* v. 62, p. 401-407.) Reviews conditions in the Culebra Cut, and the vast quantity of work caused by the earth slides there.

EARTH SLIDE AT THE NORTH DIKE OF THE WACHUSETT RESERVOIR. 1907. (*Engineering Record,* v. 55, p. 515-516.) Soil of embankment was a very fine, impermeable sand.

EXTENSIVE EARTH SLIPPAGE SHUTS DOWN CEMENT PLANT. 1915. (*Engineering News,* v. 74, p. 330-332.) Account of the successive slipping of large slices of clay soil at Greenport plant of Knickerbocker Portland Cement Company, near Hudson, N. Y. Illustrations of damaged property; diagram of area affected; description of slides; topography of the land, and nature of soil.

EXTENT AND VOLUME OF EARTH SLIDES AT CULEBRA CUT, PANAMA CANAL, and the Remedy Being Employed. 1912. (*Engineering-Contracting,* v. 38, p. 374-375.) Assigns two reasons: the sliding of one stratum on another, and the squeezing out of the softer underlying material, owing to the heavy weight above.

FERNOW, B. E. Avalanches. 1890. (*Transactions,* Am. Inst. Min. Engrs., v. 18, p. 583-597.) Considers extent, nature, and causes, protective measures, and rescue work. Chief attention to snow, comparing its action to that of sand. Advocates reforestation or construction of artificial retaining works.

FORD, FREDERICK L. Settlement of Lorraine Street, Hartford, Conn. 1902. (*Engineering Record,* v. 45, p. 172-173.) Describes progress of slow land slide and drainage method used in preventing further damage.

FORSBERG, R. P. Earth Slide at Bellevue, Penn., and Suggestions for Arresting its Further Progress. 1914. (*Engineering News,* v. 71, p. 15-18.) Cause of slide is believed to be the fact that a vein of fire-clay, thoroughly saturated with water, underlies the slide zone. The most effective solution would be to drain the clay thoroughly.

GAILLARD, D. D. Culebra Cut and the Problem of the Slides. 1912. (*Scientific American,* v. 121, p. 388-390.)

GENERAL GOETHALS ON THE PANAMA CANAL SLIDE CRITICS. 1916. (*Engineering News,* v. 76, p. 986-989.) Gives essential features of General Goethals' final statement on the slides in Gaillard Cut. Is a portion of his Annual Report to the Secretary of War.

GOETHALS, GEORGE W. Slides at Panama. 1916. (*Canal Record,* v. 9, Supplement to January 5, 1916. 17 p.) Illustrated paper describing the slides and the methods adopted for dealing with them. Contains review of experiments made in attempting to check progress of slides.

——Condensed. 1916. (*Engineering News,* v. 75, p. 417.)

GREGORY, CHARLES HUTTON. On Railway Cuttings and Embankments, with an Account of Some Slips in the London Clay, on the Line of the London and Croydon Railway. 1844. (*Minutes of Proceedings,* Inst. C. E., v. 3, p. 135-145.) Discussion, p. 145-173. Describes slips in railway cuttings caused by increased lateral pressure due to vibrations.

HAYES, C. W. Slides in Culebra Cut. 1910. (*Canal Record,* v. 4, p. 115.) Report to the President by the Chief Geologist of the United States Geological Survey. Recommends the employment of a competent geologist regularly until completion of the canal. Record of geological facts, as revealed by excavation, should be studied, and additional core drill records should be obtained. Geologist should co-operate with engineers in determining economical slopes.

HARTE, CHARLES RUFUS. Stop Slides by Releasing Accumulated Water at Bulls Ridge Hydro-electric Plant. 1916. (*Engineering Record,* v. 73, p. 696-698.) Illustrated article describing causes of and remedies for land slides that threatened the operation of a hydro-electric plant in Connecticut.

HOWARD, E. E. Making Earthwork Approach to Columbia River Bridge. 1916. (*Engineering News,* v. 75, pt. 1, p. 145-149.) Treats, in part, of a special case of earth settlement, and of the overthrow of a masonry pier by an earth slip.

HOWE, ERNEST. Landslides in the San Juan Mountains, Colorado, Including a Consideration of Their Causes and Their Classification. 55 p. 1909. (United States Geological Survey, *Professional Paper 67.*)

HUNT, E. B. On the Use of Salt-marsh Sods for Facing the Steep Slopes of Parapets, Terraces, etc. 1855. (*Proceedings,* Am. Assoc. for the Advancement of Science, v. 9, p. 272-275.) Were used successfully for parapet work of a fort at Gloucester, Mass., and at Fort Adams.

IMPROVEMENT OF A SLIDING CUT ON THE CLEVELAND, CINCINNATI, CHICAGO and St. Louis Ry. 1908. (*Engineering News,* v. 59, p. 478-479.) Improvement by better drainage.

ISAACS, JOHN D. Stopping a Troublesome Slide at a Summit Tunnel. 1895. (*Journal,* Assoc. Eng. Soc., v. 15, p. 113-123.) Concrete retaining wall, pierced by a tunnel.

LAMOTHE. Note sur les Travaux de Consolidation de la Tranchée de l'Estoura sur le Chemin de Fer de Marvejols à Neussargues. 1 pl. 1890. (*Annales des Ponts et Chaussées,* ser. 6, v. 20, p. 231-238.) Describes railway slides and methods of repair.

LANDSLIDE AT CROW'S NEST PASS. 1907. (*Engineering and Mining Journal,* v. 84, p. 1110.) Note on inspection of mountain to determine probability of land slide. Officials of the mining company report that camp is in no danger. The fissure which caused alarm is merely a widening of the natural jointage planes. The rock strata are nearly horizontal, and the slope of the mountain is less than the angle of rest.

LANDSLIDES. 1903. (*Engineering Record,* v. 48, p. 581-582.) Editorial deprecating the lack of literature on this subject in English, and suggesting a thorough acquantance with the foreign literature on slides.

LANDSLIDES ON THE BOLAN RAILWAY, INDIA. 1893. (*Engineering News,* v. 29, p. 268.) Discusses movement where a whole mountain seems to slide.

LAURENCE, W. K. Saltford Slip. 1900. (*Transactions,* Inst. Min. Engrs., v. 20, p. 476.) Landslide in side of railway cut made sixty years before. Due probably to percolation of water down to an inclined bed of limestone, perhaps furthered by vibration of passing trains.

LEFEBVRE, RENÉ. Mémoire sur la Constitution des Terres et sur les Accidents dans les Terrains Argileux. 6 dr. 1878. (*Annales des Ponts et Chaussées,* ser. 5, v. 16, p. 390-445.) Paper was translated by Lieut. F. A. Mahan. Considers slips in cuts and fills as due to pressure of contained water and dependent on the permeability and penetrability of the earths present. Studies the construction of various earths in this respect. Develops a theory of slips and discusses practical methods of prevention, giving examples of their application.

——Translation. 6 dr. 1882. (*Transactions,* Engineers' Society of Western Pennsylvania, v. 1, p. 70-105.)

LEHWALD. Die Rutschungen auf der Theilstrecke Treysa-Malsfeld (Nordhausen-Wetzlar) im Zuge der Berlin-Coblenzer Eisenbahn. 11 dr., 5 pl. 1885. (*Zeitschrift für Bauwesen,* v. 35, p. 209-231.)

LOW, EMILE. A Large Land-slide. 2 diag. 1892. (*Proceedings,* Engrs. Club of Philadelphia, v. 9, p. 245-247.) Slide in cut on Clinch Valley Division of Norfolk and Western Railroad.

——Abstract. 1892. (*Engineering and Mining Journal,* v. 53, p. 134.) Discussion by E. V. d'Invilliers.

MacDONALD, DONALD F. Landslides of Culebra Cut. 1912. (Annual Report, Isthmian Canal Commission, 1912, p. 205-214.) Names four types of slides, (1) those resulting from structural breaks and deformations, (2) normal or gravity slides, (3) fault-zone slides, (4) those resulting from weathering and surface erosion. Causes and remedies for each are discussed.

MacDONALD, DONALD F. Slides in the Culebra Cut at Panama; a Review of Geological Conditions in the Canal Site, Together with a Description of the Types of Slides and Their Causes. 1912. (*Engineering Record,* v. 66, p. 228-233.) *See also* Editorial, p. 225. Concludes that "when the slopes shall have been reduced to the proper angle . . . , the slide problem will be practically solved."

MacDONALD, DONALD F. Sliding Ground in Culebra Cut. 1913. (*Engineering News,* v. 70, p. 408.) Gives reasons why methods of preventing earth and rock slides proposed by Rice would not be applicable to Culebra Cut slides, along the Panama Canal.

MERRICK, A. W. Clay Slide at the Boone Viaduct, Boone, Iowa. 1906. (*Journal,* Western Soc. of Engrs., v. 11, p. 332-334.) Discussion, p. 335-339. Successful draining stopped the slide.

——1907. (*Proceedings,* Eighth Annual Convention, Am. Ry. Eng. and Maintenance of Way Assoc., p. 555-582.)

——1907. (Am. Ry. Eng. and Maintenance of Way Assoc., *Bulletin 88,* p. 4-11.)

METHODS AND COST OF ELECTRIC SHOVEL WORK REMOVING SLIDES AND Slide Cutting for Electric Railways. 1915. (*Engineering-Contracting,* v. 43, p. 154-155.) Includes tables of cost data.

MOLITOR, DAVID. Landslides. 1894. (*Journal,* Assoc. Eng. Soc., v. 13, p. 12-32.) Discusses and classifies slides, and works out formula for earth pressure on walls.

MORRIS, GEORGE A. Earth Slips on the Jordan Level Marl Beds of the Erie Canal. 1898. (*Engineering News,* v. 40, p. 338-339.) Describes remedy adopted.

NEWLAND, D. H. Water-Soaked Bed of Blue Clay Caused Land-Slip at Cement Plant near Hudson. 1915. (*Engineering Record,* v. 72, p. 253-254.) Gives reasons for the land-slip which caused the wreck of the power-house of the Knickerbocker Portland Cement Company's plant at Greenport, N. Y.

NEWMAN, JOHN. Earthwork Slips and Subsidences upon Public Works; Their Causes, Prevention and Reparation. 234 p. 1890.

NOVEL METHOD OF STOPPING A LANDSLIDE AT SEATTLE, WASH. 1894. (*Engineering News,* v. 31, p 387.) Method was to divert water from bed of hard, smooth clay, reducing the tendency of clay above to slide.

OFFICIAL INVESTIGATION OF THE FRANK DISASTER BY THE GEOLOGICAL Survey of Canada. 1903. (*Engineering News,* v. 49, p. 492.)

PEARCE, WILLIAM. Great Rockslide at Frank, Alberta. 1903. (*Engineering News,* v. 49, p. 490-492.)

POLLACK, VINCENZ. Ueber Seeufer-senkungen und Rutschungen. 4 pl. 1889. (*Zeitschrift,* Oesterreichischen Ingenieur- und Architekten-Vereines, v. 41, p. 5-21.) Extensive review of the literature on the subject.

PORTIER, ARSÈNE. Glissement de Terrain au Viaduc du Gor (Espagne). 1907. (*Mémoires et Compte Rendu des Travaux* de la Société des Ingénieurs Civils de France, 1907, pt. 1, p. 437-450.)

RAILWAY LANDSLIDE AT CLEVELAND. 1903. (*Engineering Record,* v. 48, p. 584.)

REPORT BY A GEOLOGIST ON SLIDES IN CULEBRA CUT AND BY A BOARD of Engineers on the Revetment of the Sides of the Cut. 1911. (*Engineering News,* v. 65, p. 21-22.)

REPORT OF THE PANAMA CANAL SLIDE COMMISSION. 1916. (*Engineering News,* v. 75, p. 599.) Abstract of the preliminary report of the Commission nominated by the National Academy of Sciences, and appointed by President Wilson, to study the Panama Canal slides.

RICE, GEORGE S. Suggested Method of Preventing Rock Slides. 1913. (*Journal,* Western Soc. of Engrs., v. 18, p. 585-602.) Discussion, p. 602-627. "Bibliography," p. 601 (24 references). Discusses and classifies types of slides as (1) those resulting from structural breaks and deformations, (2) normal or gravity slides, (3) fault-zone slides, (4) weather and surface erosion. Proposed plan for prevention provides for construction of underground retaining walls.

——Condensed. (*Engineering News,* v. 69, p. 1181.)

ROCK SLIDE AT FRANK (ALBERTA). 1903. *The Canadian Engineer,* v. 10, p. 164-166, 154.)

ROHWER, H. Discussion on Earth Slides. 1907. (Am. Ry. Eng. and Maintenance of Way Assoc., *Bulletin 90*, p. 4-10.) Emphasizes importance of selection of material to be used in making fills.

——Condensed. 1907. (*Engineering Record*, v. 56, p. 374-375.)

——Condensed. 1907. (*Engineering News*, v. 58, p. 563-564.)

——Condensed. 1907. (*Railroad Gazette*, v. 43, p. 724-726.)

RUSSELL, ISRAEL C. Landslides. 1899. (Twentieth Annual Report, U. S. Geol. Survey, pt. 2, p. 193-204.) Explains conditions of soil occurrence under which landslides are likely to take place. Refers particularly to conditions in Northern Washington.

ST. ALBAN LANDSLIDE, NEAR QUEBEC. 1894. (*Railroad Gazette*, v. 26, p. 458-459.) Description of remarkable landslide, and probable cause.

——1894. (*Scientific American Supplement*, v. 38, p. 15477-15478.)

SAVILLE, CALEB MILLS. Earth Slip in the Face of the Embankment of the North Dike of the Wachusett Reservoir. 1907. (*Engineering News*, v. 57, p. 464-465.)

SHOWALTER, WILLIAM JOSEPH. Battling with the Panama Slides. 1914. (*National Geographic Magazine*, v. 25, p. 133-153.) A descriptive rather than technical account of the difficulties encountered.

SINKING LAND WRECKS CEMENT COMPANY'S POWER PLANT. 1915. (*Engineering Record*, v. 72, p. 179-180.) Short illustrated article, with comment by D. W. Newland, Assistant New York State Geologist, on the sudden drop of supporting soil which caused the collapse of steel-frame building and 170-ft. chimney.

SLIDES ON THE PANAMA CANAL. 1911. (*Engineering News*, v. 65, pp. 570-573.) Discusses underlying causes of slides in the Culebra Cut and best means of prevention.

SMITH, FRANK B. Frank Disaster. 1903. (*Canadian Mining Review*, v. 22, p. 102-103.) Details of rock slide, Turtle Mountain, Frank, Alberta.

SMITH, R. W., and ZULCH, W. G. Solution of a Landslide Fault. 1914. (*Engineering and Mining Journal*, v. 97, p. 1090-1091.) Considers a Colorado gold district, in which an earth movement displaced the veins so that the outcrop portions were discontinuous. The sliding movement was studied by means of a topographic survey.

SOULAVY, OTTOKAR, and SCHMIDT, CARL. Ueber Eisenbahnbau-und Reconstructions-Arbeiten im Rutschterrain. 1898. (*Zeitschrift*, Oesterreichischen Ingenieur- und Architekten-Vereines, v. 50, p. 4-10, 18-22, 35-40.) Discusses nature of landslides as influenced by geological formation, and gives examples of construction work in different places.

SPENCER, J. W. Landslide at Brantford, Ontario, Illustrating the Effects of Thrusts upon Yielding Strata. 1887. (*American Naturalist*, v. 21, p. 267-269.)

SPRAGUE, N. S. Improvement of Chislett Street, Pittsburgh: Method of Supporting a Street over an Earth Slide by Using a Special Reinforced-Concrete Retaining Wall and Platform. 1914. (*Engineering Record*, v. 69, p. 389.) A construction was adopted which would be independent of ground movements. Two rows of concrete piles, 12 ft. apart, were driven into firm ground.

STANTON, ROBERT BREWSTER. Great Land-slides on the Canadian Pacific Railway in British Columbia. 1897. (*Minutes of Proceedings*, Inst. C. E., v. 132, p. 1-20.) Discussion, p. 21-46. Describes slides. Considers cause to be the irrigation water soaking downward into the silt.

——Condensed. 1898. (*Engineering*, v. 65, p. 29.)

TRATMAN, E. E. R. Foreign Railway Construction in Sliding Ground. 1906. (*Journal*, Western Soc. of Engrs., v. 11, p. 339-350.) Gives instances of earth slides along various European railways, with methods of remedying, mainly by improved drainage.

——1907. (Am. Ry. Eng. and Maintenance of Way Assoc., *Bulletin 88*, p. 12-21.)

TURTLE MOUNTAIN ROCK SLIDE. 1903. (*Engineering and Mining Journal*, v. 76, p. 10-12.) Discusses slide and structure of mountain at Frank, Alberta.

VAN HORN, FRANK R. Landslide Accompanied by Buckling, and Its Relation to Local Anticlinal Folds. 1908. (*Bulletin*, Geol. Soc. of Am., v. 20, p. 625-632.) Describes slide at Cleveland and discusses causes.

WHITLEY, HENRY MICHELL. Earthwork Slips on the Castle Eden and Stockton Railway. 2 diag. 1880. (*Minutes of Proceedings*, Inst. C. E., v. 62, p. 280-284.)

YOUNG, L. E., and STOEK, H. H. Subsidence Resulting from Mining. 205 p. 1916. (University of Illinois Engineering Experiment Station, *Bulletin 91*.) "Bibliography", p. 180-205. Exhaustive report prepared under co-operative

agreement between the University of Illinois Engineering Experiment Station, the Illinois Geological Survey, and the U. S. Bureau of Mines. Considers nature and theory of subsidence, resultant damage, protective measures, and legal considerations. Includes results of laboratory experiments. A valuable feature is the extensive classified bibliography.

ZINN, A. S. Truth about the Culebra Cut Slides, Panama Canal. 1913. (*Engineering News*, v. 70, p. 406-408.) Discusses movement of earth and means of overcoming the difficulties.

CHEMICAL AND PHYSICAL PROPERTIES OF SOILS.

THEORY.

See also **Granular Materials.**

ADAMS, FRANK D., and COKER, ERNEST G. Investigation into the Elastic Constants of Rocks, more Especially with Reference to Cubic Compressibility. 69 p. 1906. (Carnegie Institution of Washington, *Publication No. 46.*) Previously printed in part in *American Journal of Science*, v. 172, p. 95-123. Measurements were taken of the longitudinal contraction and lateral expansion under a longitudinal stress from which data all the elastic properties were estimated.

ADAMS, FRANK D., and COKER, ERNEST G. Investigation into the Elastic Constants of Rocks, more Especially with Reference to Cubic Compressibility. 6 diag., 6 dr. 1906. (*American Journal of Science*, v. 172, p. 95-123.)

——Abstract. 1907. (*Beiblätter zu den Annalen der Physik*, v. 31, pt. 1, p. 186-187.)

——Abstract. 1907. (*Minutes of Proceedings*, Inst. C. E., v. 169, p. 476-477.)

AIRY, WILFRID. On the Slopes of Cuttings. 1879. (*Minutes of Proceedings*. Inst. C. E., v. 55, p. 241-251.) Mathematical discussion on friction of soils.

AMERICAN SOCIETY OF CIVIL ENGINEERS. Progress Report of the Special Committee to Codify Present Practice on the Bearing Value of Soils for Foundations. 1915. (*Proceedings*, Am. Soc. C. E., February, 1915. Papers and Discussions, p. 491-513.) "Bibliography of physical properties and bearing value of soils," p. 497-513. Prepared by Carnegie Library of Pittsburgh.

AMERICAN SOCIETY OF CIVIL ENGINEERS. Progress Report of the Special Committee to Codify Present Practice on the Bearing Value of Soils for Foundations. 1916. (*Proceedings*, Am. Soc. C. E., March, 1916. Papers and Discussions, p. 343-367.) Discussion, May, 1916, p. 821-822. Second report, directing attention to two fundamental phases of the problem: (1) Present practice on the bearing value of soils. (2) Physical characteristics of soils in relation to engineering structures.

BASQUIN, O. H. Circular Diagram of Stress and Its Application to the Theory of Internal Friction. 1912. (*Journal*, Western Soc. of Engrs., v. 17, p. 815-847.) Discussion, p. 847-849. Gives a mathematical discussion of the internal friction of substances, using a method known as the circular diagram of stress. Also gives practical examples of the application of the theory. Pages 837-847 devoted to earth stresses and piles.

BELL, ARTHUR LANGTRY. Lateral Pressure and Resistance of Clay, and the Supporting Power of Clay Foundations. 1915. (*Minutes of Proceedings*, Inst. C. E., v. 199, p. 233-272.) Discussion, p. 272-336. Extensive, technical treatment of this one type of soil. Offers a modification of Rankine's theory, which, when applied to clay, yields results more in accordance with observed facts. Gives methods of testing used in determining properties of clay, and gives tables and diagrams listing the results obtained from tests. Gives considerable mathematical theory of the properties of clays.

——Condensed. (*Architect and Contract Reporter*, v. 93, p. 263-265.)

BRANNER, JOHN C. Structural Engineering and Earthquakes. 1915. (*Engineering Record*, v. 72, p. 780-781.) Points out that earthquakes generally are not dangerous, and shows that the danger may be further mitigated by determining the exact location of active "faults". Urges the co-operation of engineers, corporations, etc., in gathering data as to the locations of faults.

CAIN, WILLIAM. Cohesion in Earth: the Need for Comprehensive Experimentation to Determine the Coefficients of Cohesion. 1915. (*Transactions*, Am. Soc. C. E., v. 81, p. 1315-1325.) Discussion, p. 1326-1341. Shows that theories of earth pressures should consider cohesion as well as friction of the particles. Describes experiments performed to determine the coefficient of cohesion. Gives short tables in which are shown figures for coefficients of friction and of cohesion of various kinds of soil.

CAMERON, FRANK K. Dynamic Viewpoint of Soils. 1909. (*Journal of Industrial and Engineering Chemistry*, v. 1, p. 806-810.) A criticism of the static theory of soils.

CHAPERON. Observations sur le Mémoire de M. de Sazilly, Stabilité et Consolidation des Talus. 1853. (*Annales des Ponts et Chaussées*, ser. 3, v. 5, p. 225-230.)

CHENOT. Nouvelle Théorie de la Poussée des Terres. 1861. (*Comptes Rendus Hebdomadaires* des Séances de l'Académie des Sciences, v. 53, p. 718.) Short abstract. Bases his theory on principles of Coulomb.

CHENOT. Sur une Nouvelle Théorie de la Stabilité des Voûtes. 1861. (*Comptes Rendus Hebdomadaires* des Séances de l'Académie des Sciences, v. 53, p. 716-718.) Based on work of Coulomb and Poncelet.

COLIN. Recherches sur les Glissements Spontanés, Contenant l'Exposé de quelques Nouveaux Principes de Mécanique Terrestre. 1840. (*Comptes Rendus Hebdomadaires* des Séances de l'Académie des Sciences, v. 10, p. 284.) Short abstract. Endeavors to prove that an earth prism of greatest power must terminate in a cycloid.

COWLES, WALTER L. Lateral Pressure in Clay from Superimposed Loads. 1912. (*Journal*, Western Soc. of Engrs., v. 17, p. 746-750.) Deduces formula to serve as basis for determining horizontal pressure from a vertical, superimposed load.

DAWKINS, BOYD. On the Relation of Geology to Engineering. 2 diag., 2 ill. 1898. (*Minutes of Proceedings*, Inst. C. E., v. 134, p. 254-277.) "Relation of superficial accumulation to solid rocks", p. 269-272.

EARTHWORKS. 1909. (*The Engineer*, London, v. 107, p. 265.) Consider especially angle of slope desirable, and drainage.

ECKARDT, A. Die mechanischen Einwirkungen des Abbaues auf das Verhalten des Gebirges. 1913. (*Glückauf*, v. 49, pt. 1, p. 352-361, 397-403.) Studies reaction between mine workings and earth pressure, mine roofs being held up by arch action in the overlying masses.

FRANCKE, ADOLF. Erddruck. 6 diag. 1901. (*Zeitschrift für Bauwesen*, v. 51, p. 639-648.) Theory.

GOUPIL, A. Note sur la Détermination Graphique de la Poussée des Terres. 4 diag. 1888. (*Le Génie Civil*, v. 12, p. 404-405.)

HETIER. Note sur le Calcul du Profil des Murs-Barrages. 8 diag., 1 pl. 1886. (*Annales des Ponts et Chaussées*, ser. 6, v. 11, p. 615-636.) Mathematical paper.

HOUSDEN, C. E. Rapid Earthwork Calculation. 31 p. 1914. Longmans. Embodies improvements in earthwork calculation suggested by a careful reconsideration of author's "Practical Earthwork Tables". Offers new tables intended to be simpler than former ones, and yet quite as useful and correct for all usual purposes.

HUBBE. Ueber die Eigenschaften und das Verhalten des Schlicks. 1 pl. 1860. (*Zeitschrift für Bauwesen*, v. 10, p. 491-520.) Results of extended experiments are given.

KREY, H. Praktische Beispiele zur Bewertung von Erddruck, Erdwiderstand und Tragfähigkeit des Baugrundes in grösserer Tiefe. 12 diag. 1912. (*Zeitschrift für Bauwesen*, v. 62, p. 95-126.)

LAUCHLI, EUGENE. Lining Long Tunnels and Tunnels Subjected to Heavy or Eccentric Ground Pressure. 1915. (*Canadian Engineer*, v. 28, p. 111-118.) Deals primarily with tunnel design and construction, but includes a section on earth pressure as a part of such design and construction, together with formulas and tables of data on earth pressure.

LAUCHLI, EUGENE. Tunneling. 238 p. 1915. McGraw, N. Y. "Importance of Geological Surveys in Connection with Tunnel Driving", p. 1-8. "Determination of the Rock Temperature in Deeply Overlaid Tunnels", p. 120-144. "Tunnels Driven through Soft Materials—Pressure Acting on Tunnels Driven Through Soft and Cohesionless Materials", p. 163-184.

LeBLANC, CH. Examen Sommaire du Traité de la Stabilité des Constructions (1er partie) du Docteur Scheffler. 1867. (*Annales des Ponts et Chaussées*, ser. 4, v. 13, p. 139-147.)

MACEY, FRANK W. Specifications in Detail. ed. 2. 620 p. 1904. Crosby. General work on the writing of specifications. Contains information on soil characteristics, foundations, retaining walls, piles. Extensive index.

MACNEILL, JOHN. Tables for Facilitating the Calculation of Earthwork in the Cuttings and Embankments of Railways, Canals, and Other Public Works. ed. 2. 368 p. 1846.

MERRILL, GEORGE P. Treatise on Rocks, Rock-Weathering and Soils. 411 p. 1897. Macmillan. Part III, p. 173-284, covers weathering of rocks; Part IV, p. 286-292, covers transportation and redeposition of rock debris; Part V, p. 299-390, treats of the regolith, or the superficial, unconsolidated portion of the earth's crust.

MERRIMAN, MANSFIELD. Theory and Calculation of Earthwork Slopes. 10 diag. 1885. (*Engineering News*, v. 13, p. 174, 183, 199, 220-221, 237, 247, 263, 278, 295, 311.) "Literature on the subject", p. 311.

MOHLER, CHARLES K. Earth Pressures. 10 diag., 1 ill. 1910. (*Journal*, Western Soc. of Engrs., v. 15, p. 765-791.) Discussion, 13 diag., 5 ill., p. 791-827. A study of the sliding prism theory of Vauban after the graphics of Rebhann and of the analytical theory of Rankine, attempting to show lack of agreement and fallacies in the theories; also formulas and results from a new method.

——Condensed. 1911. (*Railway and Engineering Review*, v. 51, p. 441, 458-460, 1012-1014.)

——Editorial. 1910. (*Engineering Record*, v. 61, p. 744.)

MOSELEY, HENRY. On a New Principle in Statics, Called the Principle of Least Pressure. 1837. (*London and Edinburgh Philosophical Magazine and Journal of Science*, ser. 3, v. 3, p. 285-288.) Moseley's theorem served as a foundation for Rankine's theory of earth pressure.

MOSELEY, HENRY. On the Theory of Resistances in Statics. 1837. (*London and Edinburgh Philosophical Magazine and Journal of Science*, ser. 3, v. 3, p. 431-436.)

MOSELEY, HENRY. Mechanical Principles of Engineering and Architecture. 699 p. 1860. Wiley, New York. "Natural Slope of Earth", p. 412-413. "Pressure of Earth", p. 413-416.

NAGAOKA, H. Elastic Constants of Rocks and the Velocity of Seismic Waves. 1 dr. 1900. (*Philosophical Magazine*, v. 216, p. 53-68.) Reprint from Publications of the Earthquake Investigation Committee in Foreign Languages, No. 4. Tables of constants.

——Abstract. 1900. (*Beiblätter zu den Annalen der Physik*, v. 24, p. 1246.)

RANKINE, WILLIAM JOHN MACQUORN. Manual of Applied Mechanics. ed. 14. 671 p. 1895. Griffin. Includes data on earth friction, earth foundations, pressure of earth, stability of earth, retaining walls, pile-driving.

RANKINE, WILLIAM JOHN MACQUORN. Manual of Civil Engineering. ed. 24. 822 p. 1911. Griffin. Includes data on properties and theories of earth, earthworks, foundations, piles and pile-driving, caissons, coffer-dams, and retaining walls.

RANKINE, WILLIAM JOHN MACQUORN. On the Mathematical Theory of the Stability of Earthwork and Masonry. 1857. (*Proceedings*, Royal Soc. of London, v. 8, p. 60-61.) States and briefly explains the two fundamental principles on which his researches on earthwork are based.

RANKINE, WILLIAM JOHN MACQUORN. On the Stability of Loose Earth. 1857. (*Philosophical Transactions*, Royal Soc. of London, v. 147, p. 9-27.) Deduces from known laws of friction, the mathematical theory of that kind of stability which depends on the mutual friction of the parts of a granular mass devoid of tenacity.

——Abstract. 1857. (*Proceedings*, Royal Soc. of London, v. 8, p. 185-187.)

SAINT-GUILHEM. Mémoire sur la Poussée des Terres avec ou sans Surcharge. 1858. (*Annales des Ponts et Chaussées*, ser. 3, v. 15, p. 319-350.) Mathematically finds the surface of rupture and pressure of earth against walls of various forms. Extends work done by Poncelet in this field. Gives angle of repose for various substances.

SAINT-VENANT, de. Résistance des Fluides. Considérations Historiques, Physiques et Pratiques Relatives au Problème de l'Action Dynamique Mutuelle d'un Fluide et d'un Solide, Specialement dans l'Etat de Permanence Supposé Acquis par leurs Mouvements. 1886. (*Comptes Rendus Hebdomadaires* des Séances de l'Académie des Sciences, v. 103, p. 179-184.)

SAZILLY, de. Notice sur les Conditions d'Equilibre des Massifs de Terres, et sur les Revêtements des Talus. 4 pl. 1851. (*Annales des Ponts et Chaussées*, ser. 3, v. 1, p. 1-157.)

TRESCA, H. Mémoire sur l'Ecoulement des Corps Solides Soumis à de Fortes Pressions. 1864. (*Comptes Rendus Hebdomadaires* des Séances de l'Académie des Sciences, v. 59, p. 754-758.) Theory of the flow of solids under pressure.

UNITED STATES—SOILS BUREAU. Soil Survey Field Book, Field Season, 1906. 319 p. 1906. Includes data on classification of soils, analysis of soils, and an extensive description of established soil types.

TESTING.

Methods and Results.

See also **Piles, Testing.**

ARRANGEMENTS FOR SOIL-LOADING TESTS. 1 dr. 1914. (*Engineering News,* v. 72, p. 647-648.) Describes efficient methods introduced by Wm. P. Snow, Lewiston, Me., and by E. McCullough, of Chicago.

AUCHINLECK, GILBERT. Measurement of Shrinkage in Soils and Its Application in Agriculture. 1912. (*West Indian Bulletin,* v. 12, p. 50-68.) States that "shrinkage determinations may be regarded as a measurement in the laboratory of the trouble that colloidal clay causes the planter in the fields."

BAINBRIDGE, F. H. Methods and Costs of Testing for Bridge Foundations. 1908. (*Engineering–Contracting,* v. 30, p. 352-354.) Describes making of borings, to determine character of foundation. Refers particularly to work on Chicago and Northwestern Railway bridge.

BARBOUR, FRANK A. Strength of Sewer Pipe and the Actual Earth Pressure in Trenches. 2 diag., 3 dr. 1897. (*Journal,* Assoc. Eng. Soc., v. 19, p. 193-241.) Gives results of experiments.

——Abstract. 1898. (*Minutes of Proceedings,* Inst. C. E., v. 132, p. 409.)

BOERNER, FRANZ. Künstliche Fundierung des Geschäftsgebäudes für das Oberlandesgericht zu Düsseldorf. 1908. (*Beton und Eisen,* v. 7, p. 340-343, 360-364.) First part considers underlying soil strata, which were made up at successive depths of (1) filling, (2) layer of alluvial earth consisting of loose sand and fine blue sand mixed with a blue clayey material, and (3) a layer of coarse-grained gravel. Loading tests are given for the different soils.

BRIGGS, LYMAN J., and others. Centrifugal Method of Mechanical Soil Analysis. 38 p. 1904. (United States Soils Bureau, *Bulletin 24.*) Also discusses briefly other methods of mechanical soil analysis.

BRIGGS, LYMAN J. Objects and Methods of Investigating Certain Physical Properties of Soils. 1901. (*Yearbook,* United States Department of Agriculture, 1900, p. 397-410.) Intended primarily for the agriculturalist, but contains much useful information for the engineer, such as mechanical analysis, tests for moisture content, and other physical properties.

BRIGGS, LYMAN J. Some Necessary Modifications in Methods of Mechanical Analysis as Applied to Alkali Soils. 1899. (United States Department of Agriculture, *Report No. 64,* p. 173-183.) Considers disintegration of soil during analysis, apparatus and method for examining soils subject to excessive disintegration, and advantages of centrifugal method for all soils, treatment of mechanical separations after ignition, and determination of water-soluble content of soils.

CAIN, WILLIAM. Earth Pressure. 1882. (*Van Nostrand's Engineering Magazine,* v. 26, p. 89-104.) Tests formula to find how closely practice confirms theory.

CAIN, WILLIAM. Experiments on Retaining Walls and Pressures on Tunnels. 21 diag. 1911. (*Transactions,* Am. Soc. C. E., v. 72, p. 403-448.) Discussion, 6 diag., p. 449-474. Discusses a large number of experiments and gives conclusions therefrom.

——Abstract. 1911. (*Minutes of Proceedings,* Inst. C. E., v. 185, p. 398.)

CONSTRUCTION OF THE BUILDINGS, BRIDGES, PIERS, AND DOCKS AT JACKSON Park [Chicago]. 1893. (*Engineering Record,* v. 28, p. 199-201.) Gives results of numerous loading tests on Chicago soil, and values of its bearing capacity.

CROIZETTE-DESNOYERS. Mémoire sur l'Etablissement des Travaux dans les Terrains Vaseaux de Bretagne. 8 pl. 1864. (*Annales des Ponts et Chaussées,* ser. 4, v. 7, p. 273-396.) Gives experiments on the resistance of soil, p. 279-281.

CROOK, T. Method for the Mechanical Analysis of Soils. 1905. (United States Department of Agriculture, *Experiment Station Record,* v. 17, p. 340.) Modification of Schöne apparatus is used, having as its essential features (1) a constant-level water reservoir that can be adjusted to any desired height, and (2) an elutriator which is conical both above and below the cylindrical portion.

——Abstract. (*Econ. Proc.,* Royal Dublin Soc., v. 13, p. 267-280.)

DUMONT, J. Sur une Nouvelle Méthode d'Analyse Physique du Sol. 1911. (*Comptes Rendus Hebdomadaires* des Séances de l'Académie des Sciences, v. 153, p. 889-891.) Sand particles obtained by ordinary methods of elutriation were covered with a humus clay coating of complex composition. It is proposed to remove this before mechanical analysis by treatment with oxalic acid.

EMPERGER, FRITZ von. Probebelastung einer "Compressol" -pylone. 1908. (*Beton und Eisen,* v. 7, p. 49-55.) Gives tabulated results of tests.

ENGELS, H. Untersuchungen über den Seitendruck der Erde auf Fundamentkörper. 10 diag., 2 pl. 1896. (*Zeitschrift für Bauwesen,* v. 46, p. 409-432.) Methods and results of extensive experiments.

ENGELS, H. Versuche über den Reibungswiderstand zwischen strömendem Wasser und Bettsohle. 4 diag., 1 dr., 2 ill., 1 pl. 1912. (*Zeitschrift für Bauwesen,* v. 62, p. 473-488.) Describes apparatus used, discusses the experiments performed and deduces mathematical relations.

FEHR, R. B., and THOMAS, C. R. Experiments on the Distribution of Vertical Pressure in Earth. 1913. (Annual Report, Pennsylvania State College, 1912-1913. p. 81-155.) Pennsylvania State College. Engineering Experiment Station, *Bulletin No. 8.* "Bibliography of Experimental Work," p. 154-155. Tests were carried out with clear, dry river sand. Conclusions were that, at depths of 12 in. for smaller loads and 23 in. for larger loads, the percentage of transmission appeared to have a maximum value of 20. At the point where a marked change occurred in the percentage of transmission, the maximum transmission was 16 per cent.

FLETCHER, C. C. Counting Method for the Mechanical Analysis of Soils. 1911. (*Science,* v. 57, p. 495-496.) Sand is obtained by subsidence, as in regular method. Total weight of silt and clay is determined by difference. Relative quantities are then found by counting the number of silt and clay particles on a counting plate. From the relation thus established, quantities of clay and silt are determined.

FLETCHER, C. C., and BRYAN, H. Modification of the Method of Mechanical Soil Analysis. 16 p. 1912. (United States Soils Bureau, *Bulletin 84.*) Most important modification was in the method for determining quantity of clay. Clay was evaporated in enameled-ware pans and weighed without transfer to platinum dishes. A further shortening of the method was obtained by abandoning altogether direct determination of clay and obtaining its percentage by difference.

FORD, R. H. Soil-Bearing Tests Determined Loading on Chicago Track Elevation Work. 1916. (*Engineering Record,* v. 74, p. 652-653.) Describes tests to determine the bearing power of soil on which an especially heavy retaining wall was to be built. Gives results of tests.

GARDNER, FRANK D. Electrical Method of Moisture Determination in Soils. 24 p. 1898. (United States Soils Division, *Bulletin 12.*)

GREATHEAD, JOHN F. Tests of Soils and Methods of Underpinning Buildings Adjoining Subway Construction on William Street, New York. 1915. (*Engineering–Contracting,* v. 44, p. 477-481.) A rather full abstract, with illustrations, of a paper appearing in September, 1915, issue of *Public Service Record,* the official publication of the New York Public Service Commission [First District?]. Gives detailed information on methods and results of soil testing in connection with the underpinning of buildings near subway construction.

HALL, ALFRED DANIEL. Mechanical Analysis of Soils and the Composition of the Fractions Resulting Therefrom. 1904. (*Journal,* Chemical Society, v. 85, p. 950-963.) Investigates methods of mechanical soil analysis. Concludes that the preliminary treatment with acid gives a better idea of the ultimate physical constitution of the soil than that obtained by working on the raw soil.

HAYS, JAMES B. Designing an Earth Dam Having a Gravel Foundation, with the Results Obtained in Tests on a Model. 1917. (*Transactions,* Am. Soc. C. E., v. 81, p. 1-24.) Discussion, p. 25-73. Intended mainly to present the results of tests on a model constructed to scale. The model was constructed, however, to help solve a specific problem in dam construction, and considerable attention is given to the quality of the soil encountered, and to the action of this soil in the model.

HILGARD, EUGENE W. Methods of Mechanical Soil Analysis. 1887. (*Proceedings,* Eighth Annual Meeting, Society for the Promotion of Agricultural Science, p. 48-50.) Explains author's reasons for rejecting the subsidence method or "beaker elutriation", and adopting the method using his "churn elutriator."

HILGARD, EUGENE W. On Soil Analyses and Their Utility. 1872. (*American Journal of Science,* v. 104, p. 434-445.) Defends soil analysis as being of great practical value. Shows advantages that should result.

HILGARD, EUGENE W. On the Flocculation of Particles, and Its Physical and Technical Bearings. 1879. (*American Journal of Science,* v. 117, p. 205-214.) Considers question in its bearing on the mechanical analysis of soils.

HILGARD, EUGENE W. On the Silt Analysis of Soils and Clays. 2 ill. 1873. (*Proceedings,* Am. Assoc. for the Advancement of Science, v. 22, p. 54-70.) Describes his method of mechanical analysis of soils, and gives results of extensive experiments, mostly on soils of Mississippi.

——1873. (*American Journal of Science and Arts,* v. 106, p. 288-296, 333-339.)

HILGARD, EUGENE W. Report on the Methods of Physical and Chemical Soil Analysis. 1893. (United States Chemistry Division, *Bulletin 38*, p. 60-82.) Gives directions for sampling soils. Describes procedure for physical examination of soil.

HOPKINS, C. G. Rapid Method of Mechanical Soil Analysis, Including the Use of Centrifugal Force. 1899. (United States Chemistry Division, *Bulletin 56*, p. 67-68.)

JENSEN, J. NORMAN. Hardpan and Other Soil Tests. 1913. (*Engineering News*, v. 69, p. 460-463.) Tests of various Chicago soils.

KILROE, J. R. Mechanical Analyses of Soils and Subsoils by Centrifugal Action; with Notes on Treatment of Samples. 1905. (United States Department of Agriculture, *Experiment Station Record*, v. 17, p. 341.) Abstract from *Econ. Proc.*, Roy. Dublin Soc., v. 10, p. 223-230. Method is combination of those of Whitney and Bennigsen.

KING, F. H. New Method for the Mechanical Analysis of Soils. 1898. (Thirteenth Annual Report, Wisconsin Agricultural Experiment Station, p. 123-133.) A method for studying the pore space in soils, based on the laws of the flow of air through capillary tubes.

KINNISON, CHARLES S. Study of the Atterberg Plasticity Method. 1915. (United States Bureau of Standards, *Technologic Paper 46*, p. 3-18.) Deals with the Atterberg method of testing clays and classifying them according to their plasticity. This method (essentially a German one) is compared, by actual tests, with two other methods that are in more common use in the United States.

LEYGUE, L. Nouvelle Recherche sur la Poussée des Terres et le Profil de Revêtement le Plus Economique. 1885. (*Annales des Ponts et Chaussées*, ser. 6, v. 10, p. 788-1003.) Contains results of extended experiments on cohesion of earth, direction, point of application, and amount of earth pressure, and calculations and formulas for retaining walls, etc.

LOUGHRIDGE, R. H. Physical Tests of Soils. 1892. (United States Department of Agriculture, Experiment Station, *Bulletin 16*, p. 156-162.) Intended primarily for the agriculturist, but contains information of value to the engineer, regarding methods of mechanical analysis of soils.

McDANIEL, A. B., and GARVER, N. B. Pressure of Wet Concrete on the Sides of Column Forms. 1916. (*Engineering News*, v. 75, pt. 2, p. 933-936.) Tests made on columns built up in laboratory and on posts of a bridge under construction to show pressure of wet concrete on sides of forms. Describes instruments and methods for measuring pressure.

McLEAN, DOUGLAS L. Wash-Boring for the Winnipeg-Shoal Lake Aqueduct. 1914. (*Canadian Engineer*, v. 26, p. 830-832.) Presents data concerning costs of equipment and operation, and methods of sinking test holes, during severe winter weather.

MAIN, CHARLES T. Foundations. 1915. (*Transactions*, Am. Soc. Mech. Engrs., v. 37, p. 821-837.) Discussion, p. 837-843. Covers briefly the general subject of foundations. Touches on kinds of soil and methods of testing soils, with special reference to foundation work.

MAIN, CHARLES T., and SAWTELL, H. E. Pile Tests Indicate Type of Substructure for Technology Buildings. 1915. (*Engineering Record*, v. 72, p. 235-238.) Presents data on the determination of the character of soil by the use of piles of different types. Also gives tables and diagrams illustrating the test results obtained by this method in making tests for the substructure of buildings at Massachusetts Institute of Technology.

MECHANICAL ANALYSIS OF SOILS. 1906. (United States Department of Agriculture, *Experiment Station Record*, v. 18, p. 114-115.) Abstract from *Jour. Agr. Sci.*, v. 1, p. 470-474. Outlines method adopted by committee of the Agricultural Education Association.

MEEM, JAMES C. Notes and Experiments on Earth Pressures. 1912. (*Proceedings*, Engrs.' Club, Philadelphia, v. 29, p. 114-130.) Includes calculations, based mainly on experiments carried out on laboratory scale.

MEEM, JAMES C. Pressure, Resistance, and Stability of Earth. 2 diag., 12 ill., 11 dr. 1910. (*Transactions*, Am. Soc. C. E., v. 70, p. 352-388.) Discussion, p. 389-411. Gives results of practical experiments, makes deductions, and gives applications of the deductions to works carried out.

——Abstract. 1910. (*Minutes of Proceedings*. Inst. C. E., v. 182, p. 355.)

——Editorial. (*Engineering Record*, v. 61, p. 744.)

METHODS AND COSTS OF MAKING A TEST OF THE BEARING POWER OF SOIL for a Building. 1 diag., 1 dr. 1910. (*Engineering-Contracting*, v. 34, p. 31.) Test conducted by the Noel Construction Co., Chicago.

MILLER, RUDOLPH P. Standard Tests of Soil. 1912. (*Engineering Record,* v. 66, p. 112.) Letter to editor giving the test regulations of the New York Building Code for determining the sustaining power of soils.

MOYER, J. A. Distribution of Vertical Soil Pressures. 1915. (*Engineering Record,* v. 71, p. 330-332.) Tests were carried out on sand, a clay mixture, and loam.

MURRAY, J. ALAN. Mechanical Analysis of Soils; a Suggestion for a Long Tube Sedimentation Process. 1906. (*Chemical News,* v. 93, p. 40-42.) Method consisted of washing and decanting. A fairly long column of water was taken. The particles all started from the top at the same time. The separation was made according to the time at which the particles reached the bottom.

——Abstract. 1906. (*Analyst,* v. 31, p. 129.)

NEATE, CHARLES. Description of the Coffer-dam at Great Grimsby. 1850. (*Minutes of Proceedings,* Inst. C. E., v. 9, p. 1-23.) With discussion. Gives tests on bearing value of soils, based on proportion of water contained, p. 17.

NEW HENRY R. WORTHINGTON HYDRAULIC WORKS AT HARRISON, N. J. 1904. (*Engineering Record,* v. 49, p. 148-152.) *See also* letter to editor "Testing Foundations for Buildings". (*Engineering Record,* v. 49, p. 436.) Gives method of testing the bearing power of soil.

ORTH, ALBERT. Ueber mechanische und chemische Bodenanalyse. 1882. (*Berichte* der Deutschen Chemischen Gesellschaft, v. 15, p. 3025-3034.) Soil is first sifted with sieves into particles of 5, 2, 1, 0.5 and 0.2 mm. diameter. A further division is made according to the time required by different-sized particles to be deposited from a stream of running water.

OSBORNE, THOMAS B. Methods of Mechanical Soil-Analysis. 1887. (Annual Report, Connecticut Agricultural Experiment Station, 1887, p. 144-162.) Discusses and compares different methods, including those of Schöne, the Berlin-Schöne, and Schloesing and Hilgard. "Beaker" method is preferred.

PFEIFFER, K. Contributions to the Study of the Mechanical Analysis of Soils and of the Determination of Outer Soil Surface by Heat of Wetting and Hygroscopicity. 1912. (United States Department of Agriculture, *Experiment Station Record,* v. 26, p. 219-220.) Abstract from *Landw. Jahrb.,* v. 41, p. 1-55. Author believes that study of mechanical analysis of soils by sieve and sedimentation should be vigorously pursued, and that it is entirely possible to classify mineral soils on the basis of their content of finer particles. An adequate classification of fine soils would be one of two groups on the basis of current velocities of 0.02 and 7 mm., respectively.

PRECAUTIONS IN INTERPRETING RECORDS OF TEST BORINGS. 1910. (*Engineering–Contracting,* v. 33, p. 585.) Editorial.

RANDALL, FRANK A. Hardpan Test at the New Cook County Hospital. 1912. (*Journal,* Western Soc. of Engrs., v. 17, p. 725-744.) Satisfactory tests of Chicago soil.

——Condensed. 1913. (*Engineering News,* v. 69, p. 463-464.)

——Abstract. 1 dr., 1 ill. 1912. (*Engineering-Contracting,* v. 37, p. 436-437.)

SABANIN, A. Different Methods of Mechanical Analysis of Soils and the Method of Double Sedimentation with a Small Sample. 1904. (United States Department of Agriculture, *Experiment Station Record,* v. 16, p. 331.) Abstract from *Jour. Expt. Landw.,* v. 5, p. 121-123. Describes modification of Fadyeyev-Williams method. Separation of particles of soil in beakers, by double sedimentation method.

SABANIN, A. Ueber eine neue Methode der Schlämmanalyse. 1903. (*V*-Internationaler Kongress für Angewandte Chemie, v. 3, p. 896-898.) Briefly describes method using only small quantity of soil (3.75 to 4 grammes). Consists of boiling in small Erlenmeyer flask, passing through sieves and allowing to settle in cylinders.

ST. PAUL BUILDING, NEW YORK CITY. 1896. (*Engineering News,* v. 35, p. 310-312.) Includes outline of methods used and results obtained in tests to determine the character of the soil on which it was proposed to set the foundations of the building.

SCHLOESING, TH. Détermination de l'Argile dans la Terre Arable. 1874. (*Comptes Rendus Hebdomadaires* des Séances de l'Académie des Sciences, v. 78, p. 1276-1279.) Preliminary treatment is given, in which soil is treated with dilute acid, and salts and acid removed by washing. Simple sedimentation process is then used.

SCHLOESING, TH. Sur l'Analyse Mécanique des Sols. 1903. (*Comptes Rendus Hebdomadaires* des Séances de l'Académie des Sciences, v. 136, p. 1608-1613; v. 137, p. 369-374, 398-402.) Describes method for separating soil into a number of grades of fineness by observing the time required for deposition from water of a given depth, and the weight of deposits formed during successive intervals of time.

SCHMOLL v. EISENWERTH, ADOLPH. Mittheilungen über pneumatische Fundirungen, und Erfahrungsresultate über die dabei vorkommenden Reibungswiderstände. 2 pl. 1877. (*Zeitschrift,* Verein deutscher Ingenieure, v. 21, p. 433-456.) Results of extensive experiments to determine the load necessary to overcome the frictional resistance encountered by caissons in various strata, or, in case of light soil, to determine the depth to which a pier must be sunk to carry the load.

——Abstract. (Translation.) 1878. (*Minutes of Proceedings,* Inst. C. E., v. 52, p. 298-302.)

——Abstract. (Translation.) 1879. (*Van Nostrand's Engineering Magazine,* v. 20, p. 119-122.)

SHANKLAND, E. C. Chicago Foundations. 1905. (*Technograph,* v. 19, p. 5-12.) Discusses kinds of soils, with brief mention of load tests.

——1905. (*Engineering Record,* v. 52, p. 131-132.)

SMITH, J. HAMMOND. Laboratory Method of Determining Pressures on Walls and Bins. 1915. (*Proceedings,* Am. Soc. for Testing Materials, v. 15, p. 382-394.) Discussion, p. 395-398. Describes apparatus for determining the point of application, line of action, and intensity of resultant pressures, on walls and bins. Also contains results of tests.

SOIL BEARING TESTS. 1912. (*Engineering Record,* v. 66, p. 304.) Describes tests performed during the construction of the Dallas-Oak Cliff Viaduct, Dallas, Texas.

SOIL TESTS REPORTED, AND SAFE UNDERPINNING METHODS IN SAND Described. 1915. (*Engineering Record,* v. 72, p. 631-633.) Gives methods and results of soil tests, and describes underpinning used, in the construction of William Street subway excavation, New York City.

STEEL, A. A. Experiments on Earth Pressure Against Retaining Walls. 1899. (*Engineering News,* v. 42, p. 261-263.) Abstract of graduating thesis, University of Nebraska. Experiments indicate need of further investigation.

STONE, G. E., and CHAPMAN, G. H. New Method for the Approximate Mechanical Analysis of Soils. 1911. (Twenty-fourth Annual Report of Massachusetts Agricultural Experiment Station, p. 115-120.) Describes short method that gives approximately accurate results.

SUDLER, EMORY. Tests of Soil for Use in Constructing an Earth Dam for a Water-works Reservoir, Baltimore, Md. 1909. (*Engineering News,* v. 61, p. 312.) Soils tested were soft rotten rock and yellow clay. Pressure and impermeability tests were made.

TEST BORING DATA, SHOAL LAKE AQUEDUCT. 1915. (*Canadian Engineer,* v. 29, p. 321-322.) Tables of cost data of hand auger test boring for investigations of soils.

TEST-BORINGS FOR FOUNDATIONS. 1889. (*Engineering News,* v. 21, p. 324.) Describes tools and methods for making test-borings.

TESTING SOIL BELOW THE SURFACE FOR FOUNDATION LOADS. 1910. (*Engineering Record,* v. 62, p. 71.) Tests were made during construction of warehouse building in New York City by various loads on pipe resting on sand at considerable depth.

TESTING THE BEARING CAPACITY OF EARTH IN EXCAVATIONS. 1 dr. 1912. (*Engineering Record,* v. 65, p. 584.) Gives methods and results of tests of hardpan.

THOULET, J. Etude Expérimentale et Considérations Générales sur l'Inclinaison des Talus de Matières Meubles. 1887. (*Annales de Chimie et de Physique,* ser. 6, v. 12, p. 33-64.)

UNITED STATES—SOILS DIVISION. Methods of the Mechanical Analysis of Soils, and of the Determination of the Amount of Moisture in Soils in the Field. 24 p. 1896. (United States Soils Division, *Bulletin 4.*) Describes methods used by Division. Method of sampling is described. Osborne's method for mechanical analysis is used.

VAN de GREYN, E. B. Sand Testing at Denver. 1915. (*Engineering Record,* v. 71, p. 551.) Describes the inexpensive apparatus used, and the details of methods of testing sand, as devised for the Engineering Department of the City of Denver, about 1913.

WETTICH, HANS. Die Bewegung des Fördergutes im Fülbrumpf. 1915. (*Stahl und Eisen,* v. 35, p. 521-527.) Flow of vertical and horizontal layers of material was studied in models.

WHITNEY, MILTON, and BRIGGS, LYMAN J. Electrical Method of Determining the Temperature of Soils. 30 p. 1897. (United States Soils Division, *Bulletin 7.*) Wheatstone bridge method.

WHITNEY, MILTON. Some Physical Properties of Soils in Their Relation to Moisture and Crop Distribution. 1892. (United States Weather Bureau, *Bulletin 4.*) Johnson and Osborn's "beaker method" of mechanical analysis of soils, p. 31.

WINKLER, E. Versuche über den Erddruck. 2 diag., 5 dr. 1871. (*Zeitschrift,* Oesterreichischen Ingenieur- und Architekten-Vereines, v. 23, p. 255-262.) Reviews the experimental work of Gadroy (1746), Gauthey (1785), Rondelet, Woltmann, Mayniel and others, and gives results of author's own experiments.

WOOLSON, IRA H. Some Remarkable Tests Indicating "Flow" of Concrete under Pressure. 1 ill. 1905. (*Engineering News,* v. 54, p. 459.) E. P. Goodrich and others believe that almost every variety of soil discloses similar phenomena which can be explained by existence of a sort of viscosity.

Instruments.

BRIGGS, LYMAN J. Electrical Instruments for Determining the Moisture, Temperature and Soluble Salt Content of Soils. 35 p. 1899. (United States Soils Division, *Bulletin 15.*) Describes in detail the electrical soil hygrometer, thermometer, and electrolytic bridge. Gives methods of standardization and practical operation.

DEVICE FOR MAKING SUBSURFACE TESTS OF THE BEARING POWER OF SOILS, with Some Examples of Operation. 3 diag. 1910. (*Engineering–Contracting,* v. 34, p. 94.) Briefly describes the apparatus invented by John F. O'Rourke, of the O'Rourke Engineering Construction Co. of New York, and gives the tests performed by means of this apparatus on the site of the Municipal Building and a warehouse in New York.

DYNAMOMETRE POUR L'ESSAI DES TERRAIN. 6 dr. 1900. (*Le Génie Civil,* v. 36, p. 394.) Describes Rudolf Mayer's apparatus.

GOLDBECK, A. T., and SMITH, E. B. Apparatus for Determining Soil Pressures. 1916. (*Proceedings,* Am. Soc. for Testing Materials, v. 16, p. 309-319.) Describes and illustrates an apparatus for measuring pressure under earth fills or against walls.

——Condensed. 1916. (*Engineering News,* v. 76, p. 339.)

MAYER, RUDOLF. Apparatus for Determining the Safe Load upon Foundations. 1900. (*Minutes of Proceedings,* Inst. C. E., v. 142, p. 408-409.) Abstract of paper in *Schweizerische Bauzeitung,* v. 28, No. 22.

MAYER, RUDOLF. Ueber den "Fundamentprüfer." 6 ill. 1900. (*Zeitschrift,* Oesterreichischen Ingenieur-und Architekten-Vereines, v. 52, p. 673-674.) Describes the hand-operated apparatus designed by the author.

MAYER, RUDOLF. Ueber einen Apparat und ein Verfahren zur Ermittlung der Tragfähigkeit des Baugrundes. 1 diag., 1 dr. 1896. (*Zeitschrift* des Oesterreichischen Ingenieur-und Architekten-Vereines, v. 48, p. 588-589.) Apparatus designed by the author.

MOYER, J. A. Distribution of Vertical Soil Pressures; Dry-Sand Tests Recently Completed at the Engineering Experiment Station of the Pennsylvania State College. 1914. (*Engineering Record,* v. 69, p. 608-609.) A platform-scale apparatus was used for determining the downward-pressure.

NEW INSTRUMENT TESTS SANDS QUICKLY IN THE FIELD. 1915. (*Engineering Record,* v. 71, p. 821-822.) Describes construction and operation of a new sand-testing device for use in the field.

SCHONE, EM. Ueber einen neuen Apparat für die Schlämmanalyse. 1868. (*Zeitschrift für Analytische Chemie,* v. 7, p. 29-47.) Describes apparatus and method for the mechanical analysis of soils.

TESTS OF SETTLEMENTS OF HEAVY LOADS ON SOIL. 1897. (*Engineering Record,* v. 35, p. 332.) Briefly describes Meyer's apparatus for measuring settlement produced by test loads on small areas of soil.

YODER, P. A. New Centrifugal Soil Elutriator. 1905. (United States Department of Agriculture, *Experiment Station Record,* v. 16, p. 448-450.) Abstract from Utah Agricultural Experiment Station, *Bulletin 89.* Describes apparatus combining principles of centrifugal and elutriator methods.

BEARING VALUE.

See also **Piles.**

AMERICAN RAILWAY ENGINEERING ASSOCIATION. Report [of Committee] on Unit Pressures Allowable on Road-beds of Different Materials. 1912. (*Proceedings,* Thirteenth Annual Convention, Am. Ry. Eng. Assoc., p. 388-396.) Discusses bearing value of soils of different kinds, as found by previous investigators.

ANGLIN, S. Foundations of Buildings. 1899. (*Journal*, Royal Inst. of British Architects, v. 49, p. 14-18.) Paper before Manchester Society of Architects. Considers what foundation soils may be looked on as reliable substrata. Discusses bearing values of different clays, gravels, and sands.

——1900. (*Stone*, v. 21, p. 243-250.)

——Abstract. 1899. (*Engineering Record*, v. 40, p. 679-680.)

APJOHN, JAMES HENRY. Note on the Movement of the Walls of the Kidderpur Docks. 4 dr. 1895. (*Minutes of Proceedings*, Inst. C. E., v. 121, p. 104-151.) With discussion. The precarious nature of clay soil is brought out in discussion by W. R. Galbraith, Sir Douglas Fox, and F. E. Robertson, p. 114, 125, 149.

BAKER, IRA O. Treatise on Masonry Construction. ed. 8. 556 p. 1898. (Section on "Bearing Power of Soils", p. 188-199. Considers different kinds of soils, and gives figures and calculations, and means for improving the bearing value. "Bearing Power of Piles", p. 233-250. "Retaining Walls", p. 338-352.

BARTLETT, JAMES. Foundations. 1910. (Encyclopædia Britannica, ed. 11, v. 10, p. 738-743.) Discusses safe loads for soils of different characters, and making of trial borings.

BAUMANN, FREDERICK. Foundations. 1898. (*Inland Architect*, v. 32, p. 42-45.) Gives figures and estimates for safe loads for rock, gravel, gravel and clay, sand, and clay. Considers nature of Chicago soil and adequate loads.

BEARING CAPACITY OF EARTH FOUNDATION BEDS. 1906. (*Engineering Record*, v. 54, p. 647-648.) Editorial comment on work done by Corthell on pressures on foundation beds of certain structures. *See also* Corthell, Elmer Lawrence.

BEARING TESTS FOR HEAVY FOUNDATION LOADS. 1 dr. 1909. (*Engineering Record*, v. 60, p. 55.) Describes method and gives results of test of the supporting power of soil composed of fine, compact sand, together with small pebbles and gravel.

BOULNOIS, W. A. On the Foundations of Some of the Metropolitan Bridges in the River Thames. 1857. (*Papers*, Royal Inst. of British Architects, v. 8, p. 31-44.) Bearing power of London clay, p. 33, 39, 42.

CAMBRIA STEEL COMPANY. Safe Bearing Capacity of Soils, etc., Tons per Square Foot. 1914. (Cambria Steel Company's "Handbook of Information Relating to Structural Steel", ed. 11, p. 310.) Table giving bearing values of different soils for twenty-nine different cities in the United States.

CLEGG, SAMUEL. On Foundations, Natural and Artificial. 1851. (*Minutes of Proceedings*, Inst. C. E., v. 10, p. 317-320.) Touches on bearing value of various soils.

CORTHELL, ELMER LAWRENCE. Allowable Pressures on Deep Foundations. 98 p. 1907. Wiley. Bibliography, p. 37-40. Contains great quantity of information and tabulated data showing allowable ranges of pressure of structures on fine sand, coarse sand and gravel, sand and clay, alluvium and silt, hard clay, and hardpan. Contains, also, letters from various engineers giving information from projects under personal observation of writers, and abstracts of many technical papers on foundations.

——Abstract. 1906. (*Minutes of Proceedings*, Inst. C. E., v. 165, p. 249-251.)

——Abstract. 1906. (*Engineering News*, v. 56, p. 657-658.)

——Abstract. 1906. (*Engineering Record*, v. 54, p. 629.) *See also* editorial "Bearing Capacity of Earth Foundation Beds." (*Engineering Record*, v. 54, p. 647-648.)

CRANDALL, CHARLES LEE, and BARNES, FRED ASA. Railroad Construction. 321 p. 1913. Contains brief treatment of bearing values of different soils (p. 156), retaining wall design (p. 201), etc.

CRUTTWELL, GEORGE EDWARD WILSON. Foundations of the River-Piers of the Tower Bridge. 1893. (*Minutes of Proceedings*, Inst. C. E., v. 113, p. 117-150.) With discussion. Gives reasons for limiting pressure on the London clay to 4 tons per sq. in., p. 148-149.

CUNNINGHAM, BRYSSON. Dock and Harbour Engineers' Reference Book. 319 p. 1914. Griffin. Data on foundations for quay and dock walls, earth pressure, angles of repose, and bearing values of different soil materials, p. 145-152.

DON SECTION, BLOOR STREET VIADUCT, TORONTO. 1914. (*Canadian Engineer*, v. 27, p. 581-584.) Notes on the preliminary work attending the whole project, foundation tests, features of design. Is principally concerned with the design of the bridge proper, but includes data on tests of bearing power of the soil involved.

FOUNDATION LOADS. 1911. (*The Builder,* London, v. 101, p. 332-333, 453-454, 780-781.) Gives figures to show pressures of structures on different earth soils at which no settlement was observed. Includes also compressive resistance of different stones, and discusses foundations under differing soil conditions.

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HARRISON, THOMAS ELLIOT. On the Tyne Docks at South Shields; and the Mode Adopted for Shipping Coals. 1859. (*Minutes of Proceedings,* Inst. C. E., v. 18, p. 490-503.) Tells of experiments on bearing value of mud or slake, p. 493.

HERMANY, CHARLES. Foundations of the New Capitol at Albany, N. Y. 1873. (*Transactions,* Am. Soc. C. E., v. 2, p. 287-289.) Gives maximum pressure per square foot.

HUNT, RANDELL. Supporting Power of Soils. 1888. (*Journal,* Assoc. Eng. Soc., v. 7, p. 189-196.)

——Abstract. 1888. (*Minutes of Proceedings,* Inst. C. E., v. 94, p. 327-328.)

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LEONARD, HUGH. Supporting Power of Rocks and Soils; Experiments at Calcutta. 1900. (*Journal,* Royal Inst. of British Architects, v. 49, p. 390-393.) Tabulated data from results of experiments, showing supporting power of ordinary undisturbed alluvial soil of Calcutta. The successive formations of earth in and about Calcutta, where tests were carried out are, rich soil, dry bluish clay, dry brownish clay, brownish clay mixed with sand and not so dry, impure peat, wet sand, and clay. Blocks of masonry were laid on foundations of various depths up to 77 ft., and were loaded and the settlement observed.

MAGINNIS, OWEN B. Consideration of the Earth's Surface in Its Relation to Building Construction. 1907. (*Architects' and Builders' Magazine,* v. 9, p. 82-84.) General discussion of value of different soil materials in foundation work and their bearing values. Very little quantitative information given.

MILLER, RUDOLPH P. Allowable Pressure on Hardpan. 1910. (*Engineering Record,* v. 62, p. 783.) Letter to editor proposing a rule for allowable pressure on hardpan.

——1910. (*Engineering News,* v. 64, p. 727.)

PNEUMATIC CAISSON FOUNDATIONS, WHITEHALL BUILDING, NEW YORK. 1 dr. 1910. (*Engineering Record,* v. 61, p. 792-794.) Gives tests of bearing power of hardpan, p. 792-793.

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WADDELL, J. A. L. A Study in the Designing and Construction of Elevated Railroads, with Special Reference to the Northwestern Elevated Railroad and the Union Loop Elevated Railroad of Chicago, Ill. 1897. (*Transactions*, Am. Soc. C. E., v. 37, p. 308-360.) Gives bearing value of Chicago soil, p. 321.

WALMISLEY, A. T. Foundations as Applied to London Buildings and Riverside Foundations. 17 dr. 1898. (*The Builder*, London, v. 74, p. 514-521.) Shows soil strata underlying London by examination of borings from various localities. Gives much information on safe bearing power of various kinds of ground. Includes also table showing angle of repose for different materials.

WILLMANN, L. von. Tragfähigkeit des Baugrundes. 1 diag., 4 dr. 1906. (Handbuch der Ingenieurwissenschaften, 4 Aufg., 1 Teil, 3. Band, p. 12-20.)

WORCESTER, JOSEPH R. Boston Foundations. 6 folding diag., 2 folding dr., 1 folding map. 1903. (*Journal*, Assoc. Eng. Soc., v. 30, p. 285-302.) Discussion, p. 302-335. Information is given both in paper and in discussion concerning allowable pressure on soils. Tabulated data are included on kinds of soils and soil combinations to be found in different parts of Boston.

——Condensed. 1903. (*Engineering News*, v. 49, p. 136.)

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CAUSE OF CAVE-IN OF RANDOLPH STREET, CHICAGO. 1913. (*Engineering Record,* v. 68, p. 715-716.) Abstract of report by a commission of municipal engineers. Inadequate provision for earth pressure.

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HAZARD, J. Judging the More Important Physical Properties of Soils on the Basis of Mechanical Analysis. 1905. (United States Department of Agriculture, *Experiment Station Record,* v. 16, p. 857.) Abstract from *Landw. Vers. Stat.,* v. 60, p. 449-474. Author attempts to trace relation between size and physical character of particles and the fertility of soil.

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MOLESWORTH, GUILFORD. [Physical Properties of Black Cotton Soil.] 5 diag. 1898. (*Minutes of Proceedings,* Inst. C. E., v. 132, p. 209-213.) In discussion on W. L. Strange's paper: "Reservoirs with High Earthen Dams in Western India."

MORRIS, ELLWOOD. On the Compression of Earth, and the Increase of Rock in Embankment, Compared with the Volume in Excavation. 1841. (*Journal,* Franklin Inst., v. 32, p. 236-240.)

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TOPOGRAPHY OF THE BED ROCK UNDERLYING CHICAGO. 1914. (*Engineering-Contracting,* v. 42, p. 601-607.) Gives information about a contour map of Chicago with a contour interval of 10 ft., based on all records that could be obtained.

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UNITED STATES—SOILS BUREAU. Field Operations of the Division of Soils, Reports [1st]—date, 1899—date. 1900—date. First report is *Report 64* of the Department of Agriculture. Although primarily undertaken to provide the agriculturist with accurate knowledge of the soils, it contains much information of value to engineers and builders.

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GRANULAR MATERIALS.

SAND AND GRAVEL.

See also **Retaining Walls.**

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BOUSSINESQ, J. Note on G. H. Darwin's Paper "On the Horizontal Thrust of a Mass of Sand." 1882. (*Minutes of Proceedings*, Inst. C. E., v. 12, p. 262-271.) Attempts to show that Darwin's conclusions are very similar to his own as expounded in his "Essai Théorique sur l'Equilibre des Massifs Pulvérulents" (Paris, Gauthier-Villars. 1876). Refers to Darwin's paper in *Minutes of Proceedings*, Inst. C. E., v. 71, p. 350.

BRAUNING, C. Gravel as Ballast. 1912. (*Proceedings*, Thirteenth Annual Convention, Am. Ry. Eng. Assoc., v. 13, p. 267-289.) Translated from *Zeitschrift für Bauwesen*, 1904. Considers bearing value of gravel particles.

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CARTWRIGHT, H. H. Tests of Grouting Gravel in River Beds. 1913. (*Engineering News*, v. 69, p. 979-984.) Experiments indicated formation of hard foundation strata to be feasible, by forcing cement grout into soft foundation gravel.

CHAPMAN, CLOYD M., and JOHNSON, NATHAN C. Economic Side of Sand Testing. 1915. (*Engineering Record*, v. 71, p. 734-737.) Letter containing correction to above, p. 813. Emphasizes the importance and the practical value of the testing of sand that is to be used in concrete. Outlines methods and cites examples showing saving effected.

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FORCHHEIMER, PH. Ueber Sanddruck und Bewegungserscheinungen im innern Trockenen Sandes. 7 diag., 2 dr., 1 pl. 1882-83. (*Zeitschrift*, Oesterreichischen Ingenieur– und Architekten-Vereines, v. 34, p. 111-126; v. 35, p. 103-108.) Gives a brief historical review of the work done on sand pressure, lateral movement of sand, and angle of repose of sand masses. Describes author's own experiments, giving the kinds of sand used, apparatus, methods of measurement and results obtained.

GAUDARD, JULES. Note on Mr. G. H. Darwin's Paper "On the Horizontal Thrust of a Mass of Sand." 1882. (*Minutes of Proceedings*, Inst. C. E., v. 72, p. 272-274.)

HERSHAM, ERNEST A. Flow of Sands Through Orifices. 1914. (*Journal*, Franklin Inst., v. 177, p. 419-444.) Experiments to determine flow of dry sands and similar substances through orifices, under varied conditions and different sand-heads. Results shown with charts and tables.

HUBBE. Von der Beschaffenheit und dem verhalten des Sandes. 2 pl. 1861. (*Zeitschrift für Bauwesen*, v. 11, p. 19-42, 183-226.)

KICK, FR. Das Gesetz der proportionalen Widerstände und seine Anwendung auf Sanddruck und Sprengen. 2 diag. 1883. (*Dingler's Polytechnisches Journal*, v. 250, p. 141-145.)

McCULLOUGH, F. M. Local Sands and Gravels as Aggregates in Concrete. 1915. (*Proceedings*, Engrs.' Soc. of Western Pennsylvania, v. 30, p. 334-367.) Discussion, p. 368-379. Concerned mainly with behavior of sand and gravel in concrete, but discusses briefly the mechanical and physical properties of sands and gravels of the Pittsburgh region.

MARSH, GEORGE P. The Earth as Modified by Human Action; a Last Revision of "Man and Nature." 629 p. 1885. Scribner. Chapter 5 ("The Sands", p. 525-583) deals with nature and distribution of sand and sand dunes.

MERRIMAN, MANSFIELD. On the Theories of the Lateral Pressure of Sand Against Retaining Walls. 1887. (*School of Mines Quarterly,* v. 9, p. 109-112.) Paper read before the American Association for the Advancement of Science. Reviews accepted formulas and claims that they are not sufficiently based on experiment.

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MONTGOMERY, CHARLES M. Sand Testing at New York. 1915. (*Engineering Record,* v. 71, p. 551-552.) Discusses test requirements for acceptance of sand used by Board of Water Supply of New York.

MURPHY, E. C. Density and Draining Capacity of Artificial and Natural Mixtures of Sand and Gravel. 1909. (*Engineering News,* v. 62, p. 335.)

PARTIOT. Mémoire sur les Sables de la Loire. 1871. (*Annales des Ponts et Chaussées,* ser. 5, v. 1, p. 233-292.)

ROLLAND, G. Sur les Grandes Dunes de Sable de Sahara. 1881. (*Comptes Rendus Hebdomadaires* des Séances de l'Académie des Sciences, v. 92, p. 968-971.)

SAND FOUNDATIONS FOR HIGH BUILDINGS. 1912. (*Engineering Record,* v. 66, p. 310.) Gives various loads on sand.

SINGULAR PHENOMENON IN SAND. 1884. (*Engineering News,* v. 11, p. 65.) Short note. Pile embedded upright in sand has been seen to move bodily down stream.

STANDARD APPARATUS AND PROCEDURE RECOMMENDED FOR SAND ANALY-sis. 1915. (*Engineering Record,* v. 71, p. 644.) Discussion by Allen Hazen, p. 644-646. Abstract of paper by Philip Burgess before American Water Works Association, May, 1915. Outlines difficulties encountered in making tests of sand for filters, for concrete, and for asphalt mixtures. Suggests remedies.

SUBSTRUCTURE OF THE NEW YORK MUNICIPAL BUILDING. 1910. (*Engineering Record,* v. 62, p. 57-58.) Editorial, chiefly discussing the tests on the bearing value of sand.

WILMS, W. H. Stripping of Gravel Pits by Hydraulic Methods. 1915. (*Railway Age Gazette,* v. 58, p. 1430-1433.) Outline of conditions under which this practice is economical, and description of the manner of handling the work.

WILSON, GEORGE. Some Experiments on Conjugate Pressures in Fine Sand, and Their Variation with the Presence of Water. 4 diag., 2 dr. 1902. (*Minutes of Proceedings,* Inst. C. E., v. 149, p. 208-222.) Experiments on lateral pressure of earth work.

QUICKSAND.

BRIGDEN, W. W. Quicksand Excavation at Battle Creek [Mich.]. 1914. (*Engineering Record,* v. 69, p. 163-164.) Paper before Michigan Engineering Society. Describes difficult piece of excavation in quicksand, in constructing a pumping station for an Artesian-well water-supply.

BRUNLEES, JAMES. Description of the Iron Viaducts Erected Across the Tidal Estuaries of the Rivers Leven and Kent, in Morecambe Bay, for the Ulverstone and Lancaster Railway. 1 pl. 1858. (*Minutes of Proceedings,* Inst. C. E., v. 17, p. 442-447.) Treats of bearing value of quicksands.

BUILDING AND MACHINERY FOUNDATIONS IN QUICKSAND. 2 dr. 1906. (*Engineering Record,* v. 53, p. 247-248.) Fourteen-story office building, with pile foundation.

LANDRETH, WILLIAM B. Improvement of a Portion of the Jordan Level of the Erie Canal. 1900. (*Transactions,* Am. Soc. C. E., v. 43, p. 566-602.) Paper and discussion giving valuable information on quicksand and its bearing value.

——Abstract. 1900. (*Engineering Record,* v. 41, p. 137.)

SINKING MACHINERY FOUNDATIONS IN QUICKSAND WITHOUT EXCAVATION. 1905. (*Engineering Record,* v. 52, p. 526.) Describes erection of boring mill in General Electric Company's plant at Schenectady.

STEEL PILE FOUNDATION IN A QUICKSAND POCKET. 1908. (*Engineering Record,* v. 57, p. 203.) Details of work on foundation of 16-story building in New York City.

GRAIN.

See also **Retaining Walls.**

AIRY, WILFRID. Pressure of Grain. 1897. (*Minutes of Proceedings,* Inst. C. E., v. 131, p. 347-358.) Considers grain as semi-fluid. Confirms formulas of Roberts.

GRANDVOINNET, J. A., and RICHOU, G. De la Pression des Grains en Magasin. 1883. (*Le Génie Civil,* v. 3, p. 109-111.) Reviews the experimental work on this subject, especially that of Isaac Roberts.

JAMIESON, J. A. Grain Pressures in Deep Bins. 1903. (*Transactions*, Can. Soc. C. E., v. 17, p. 554-654.) Reviews previous work and describes experiments with grain in the bins of the Canadian Pacific Railway elevator at West St. John, N. B.

—— 1904. (*Engineering News*, v. 51, p. 236-243.)

JANSSEN, H. A. Versuche über Getreidedruck in Silozellen. 1895. (*Zeitschrift*, Verein deutscher Ingenieure, v. 39, p. 1045-1049.) Tests to obtain vertical pressure of grain. After obtaining the coefficient of friction between grain and the bin wall materials, author derived a formula for calculating pressures in bins of different sizes.

——Condensed. 1896. (*Minutes of Proceedings*, Inst. C. E., v. 124, p. 553-555.)

LUFFT, ECKHARDT. Tests of Grain Pressure in Deep Bins at Buenos Aires, Argentina. 1904. (*Engineering News*, v. 52, p. 531-532.) Tests were made on a bin of 19 000 bushels capacity. Satisfactory agreement was obtained between the calculated and observed pressures.

——Abstract. 1905. (*Le Génie Civil*, v. 46, p. 214-215.)

PLEISSNER, J. Versuche zur Ermittlung der Boden- und Leitenwanddrücke in Getreidesilos. 1906. (*Zeitschrift*, Verein deutscher Ingenieure, v. 50, p. 976-986, 1017-1022.) Experimental determination of the actual vertical and lateral pressures of grain in wooden and reinforced concrete bins.

——Abstract. 1906. (*Le Génie Civil*, v. 49, p. 271-272.)

PRANTE. Messungen des Getreidedruckes gegen Silowandungen. 1896. (*Zeitschift*, Verein deutscher Ingenieure, v. 40, p. 1122-1125.) Tests were made with a view to obtaining the lateral pressure of grain in a cylindrical bin. Greatly increased pressure obtained with grain in motion, when being drawn out of the bin.

ROBERTS, ISAAC. Pressure of Stored Grain ; on the Pressure of Wheat Stored in Elongated Cells or Bins. 1882. (*Engineering*, v. 34, p. 399.) Paper before British Association for the Advancement of Science.

SEE, ALEXANDRE. Calcul des Parois des Silos a Grains. 8 diag. 1905. (*Le Génie Civil*, v. 46, p. 377-379.) Develops a mathematical theory of grain pressure on container walls.

WHITED, WILLIS. Flow of Semi-fluids Through Orifices. 1901. (*Proceedings*, Engrs.' Soc. of Western Pennsylvania, v. 17, p. 113-129.) Investigates flow of solid particles, somewhat spherical in form, and without adhesion. Advances no theory to explain results obtained. Wheat was used in carrying out experiments.

MISCELLANEOUS.

COMPUTATION OF STRAINS IN COAL AND GRAIN BINS. 1897. (*Engineering News*, v. 38, p. 200-201.) Discusses pressure of solids upon sides of bins. *See also* Letters, p. 293-296, 426-428.

COYKENDALL, T. C. Theory of Stress in a Granular Mass. 6 diag. 1890. (*School of Mines Quarterly*, v. 12, p. 1-13.) Mathematical paper.

DESIGN OF THE COAL BIN AT THE PATERSON ELECTRIC LIGHT STATION. 1897. (*Engineering News*, v. 38, p. 196-198.) Discusses lateral pressure of coal against sides of bin, that had failed. Includes letter from the designers on the principles of design followed.

DULL, R. W. Some Formulas and Tables for Bin Designing. 1904. (*Engineering News*, v. 52, p. 62-66.) Considers pressures on bin walls, with particular reference to pressure of bituminous coal.

FAILURE OF A LARGE COAL BIN AT PATERSON, N. J. 1897. (*Engineering News*, v. 38, p. 142-144.) Does not go into details of design.

KETCHUM, MILO S. Design of Walls, Bins and Grain Elevators. ed. 2. 556 p. 1911. Pt. 1, p. 3-163, is on "The Design of Retaining Walls." Discusses work of previous investigators, and draws conclusions from it. Exhaustive review of experiments on pressures of grain and other materials in bins.

LATERAL PRESSURE OF GRANULAR MATERIALS. 1904. (*Engineering Record*, v. 49, p. 502.) Editorial discussion of experimental work by Jamieson with grain, and by Goodrich with earth.

RANKINE, WILLIAM JOHN MACQUORN. On the Stability of Loose Earth. 4 diag. 1857. (*Philosophical Transactions*, Royal Soc. of London, v. 147, pt. 1, p. 9-27.) "Mathematical theory of that kind of stability, which, in a mass composed of separate grains, arises wholly from the mutual friction of those grains, and not from any adhesion amongst them."

——Translated. 1874. (*Annales des Ponts et Chaussées*, ser. 5, v. 8, p. 131-168.)

ROBERTS, ISAAC. Determination of the Vertical and Lateral Pressures of Granular Substances. 1884. (*Proceedings*, Royal Soc. of London, v. 36, p. 225-240.) Experiments were carried out with wheat and peas, with the object of determining pressures, to be used in calculations for the design of bins.

SMITH, J. HAMMOND. Laboratory Method of Determining Pressure on Walls and Bins. 1915. (*Proceedings*, Am. Soc. for Testing Materials, v. 15, pt. 2, p. 382-394.) Discussion, p. 395-398. Describes apparatus for testing pressure on walls and bins, explains theory of apparatus, and presents test results in tabulated form.

FOUNDATIONS

See also **Chemical and Physical Properties of Soils. Bearing Value. Piles.**

ABBOTT, HUNLEY. Method of Constructing Gas Holder Foundations in Soft Soils, with Some Costs. 1912. (*Engineering–Contracting*, v. 37, p. 199-200.)

AKADEMISCHER VEREIN HUTTE. Hütte, des Ingenieurs Taschenbuch. ed. 21. 3 v. 1911. "Statik der Baukonstruktionen", v. 3, p. 56-225; "Grundbau", v. 3, p. 226-264. Includes consideration of earth pressure and retaining wall design.

ALLAIRE, ALEXANDER. Foundation Work in Montreal. 1 dr., 4 ill. 1912. (*Canadian Engineer*, v. 22, p. 188-190.) Considers, in general, quality of soils underlying Montreal and types of foundations constructed for different structures.

AMERICAN RAILWAY ENGINEERING ASSOCIATION. Design of Foundations for Piers, Abutments, Retaining Walls and Arches in Various Soils and Depths of Water (not Including Pneumatic Foundations). 1916. (*Proceedings*, Seventeenth Annual Convention, Am. Ry. Eng. Assoc., v. 17, p. 225-231.) Report of Committee VII, Sub-Committee "D". Considers bearing value of soils for foundations; allowable pressure on foundation soils and on piles; uplift or buoyancy effect on submerged foundations; formulas for determining pressures on foundations; pile cut-off and proper depth of foundation to avoid frost action.

ANDERSON, WILLIAM. Antwerp Water Works. 1883. (*Minutes of Proceedings*, Inst. C. E., v. 72, p. 24-44.) Discussion, p. 44-78. Foundations, p. 37-38, 53-55. Different ways of building foundations are discussed, also supporting power of piles as calculated by Sanders' formula.

AUS, GUNVALD. Reinforced Wall Foundations on Yielding Subsoil. 800 w. 3 dr. 1908. (*Engineering News*, v. 60, p. 5.) Briefly touches on yielding properties of light soils, and favors use of continuous foundation under walls, heavily reinforced with longitudinal and transverse steel or reinforced concrete beams.

BASSELL, BURR. Earth Dams: A Study. 66 p. 1904. Careful study of principles involved in building dams, with descriptions of important dams. Contains chapter, "Outline Study of Soils."

BLIGH, W. G. Dams and Weirs. 206 p. 1915. American Technical Society. Contains considerable data on the influence of the character of foundation soils, with special reference to dams and weirs.

BLIGH, W. G. Practical Design of Irrigation Works. ed. 2, rev. and enl. 443 p. 1910. Van Nostrand. Chapter 6, p. 162-205, treats of "Diversion Weirs on Sand Foundations." Devoted to the construction and performance of weirs having no foundation other than sand.

BLYTH, EDWARD LAWRENCE IRELAND. Description of the Loch Ken Viaduct, Portpatrick Railway. 1862. (*Minutes of Proceedings*, Inst. C. E., v. 21, p. 258-264.) Presents data on load supported on piers, p. 260.

BOWEN, CHARLES F. Difficult Foundation Work on the Amoskeag Savings Bank Building, Manchester, N. H. 1914. (*Engineering News*, v. 71, p. 1126-1129.) Difficulty caused by water-bearing strata below basement level, and by presence of adjacent party wall.

BRICK AND IRON FOUNDATION FOR A GERMAN BUILDING. 800 w. 1898. (*Engineering Record*, v. 38, p. 9.) Alluvial clay stratum with supporting power of about 3 200 lb. per sq. ft. makes up foundation soil.

BUETTELL, R. B. Foundations of the Union National Bank Building, Cleveland, Ohio. 1916. (*Wisconsin Engineer*, v. 20, p. 268-273.) Gives details of difficulties encountered in designing foundations for this building, and, in some detail, treats of the influence of the character of the soil on such design.

BUILDING POWER HOUSE AND DAM ON SAND FOUNDATION. 1916. (*Engineering News*, v. 75, p. 1113-1116.) Power-house and 1 000-ft. concrete dam built on deep sand bed of Wisconsin River. Sand confined between lines of steel sheeting and covered with loose rock. Wood piles driven within the inclosure. Special protection against wash at down-stream side of dam.

CHRISTENSEN, C. L. Design of Pole Foundations; Derivation of Formulæ Based on the Compressive and Frictional Resistance to Overturning Offered by Ordinary Soil. 1914. (*Engineering Record*, v. 70, p. 243-244.) Deals with design of foundations for transmission line poles. Considers resisting moment from passive earth pressure.

COMPOSITE SAND AND ROCK FOUNDATION FOR A TALL BUILDING. 2 dr. 1910. (*Engineering News*, v. 63, p. 24-26.) Describes foundation base of Municipal Building, New York City, 25-story office building with a tower rising 560 ft. above street level. Decision is for two-thirds of building to be founded on rock, the remainder on sand.

CONCRETE FOUNDATIONS IN SHIFTING GROUND. 1908. (*Engineering News*, v. 59, p. 573.) Stable foundations were obtained by founding chain-conveyor footings on rock below earth which had been gradually creeping or sliding.

CONSTRUCTING THE FOUNDATIONS OF THE SEAMEN'S CHURCH INSTITUTE, New York. 1912. (*Engineering Record*, v. 65, p. 105-107.) Pneumatic wall caissons were sunk through mud, clay, sand, and hardpan, to about 40 ft. below street.

CONSTRUCTION D'UN GRAND BATIMENT INDUSTRIEL EN BETON ARME SUR UN Terrain sans Résistance. 1910. (*Le Génie Civil*, v. 56, p. 454.) Short note giving Italian practice.

CORRIVEAU, R. de B. Settlement of a Foundation on Silt and Alluvium. 1907. (*Engineering News*, v. 57, p. 71.) Letter to editor. Gives case of settlement of a shallow foundation in silt and alluvium.

CREMERS, J. J. CANTER. Détermination des Eléments Necessaires au Calcul de la Poussée des Terres. 1916. (*Le Génie Civil*, v. 69, p. 75-77.) Calculation of earth pressures with reference to the use of concrete in foundations.

DANTIN, CH. Calcul des Dimensions et du Pouvoir Porteur des Pieux de Fondation. 6 diag. 1912. (*Le Génie Civil*, v. 60, p. 246-250.) Mostly a review of the work by Benabeng and Resal on the subject.

DESNOYER, CROIZETTE. Mémoire sur l'Etablissement des Travaux dans les Terrains Vaseaux de Bretagne. 8 pl. 1864. (*Annales des Ponts et Chaussées*, ser. 4, v. 7, p. 275-396.) French foundation practice on alluvial soils.

DETERMINATION EXPERIMENTALE DE LA RESISTANCE DES TERRAINS DE Fondation. 1908. (*Le Génie Civil*, v. 53, p. 293-294.)

DEVELOPMENT OF SHALLOW AND DEEP FOUNDATIONS FOR CHICAGO BUILD-ings. 1904. (*Engineering News*, v. 52, p. 560-563.) Gives brief information as to nature of Chicago soils at different depths.

DILLEY, WILFRID JOSEPH. Footings in Foundations. 6 diag. 1905. (*Minutes of Proceedings*, Inst. C. E., v. 163, p. 309-318.) Discusses tangential stress on the soil, and stresses and reactions when center of pressure coincides with geometrical center of area of foundation.

DOBSON, E. Foundations and Concrete Works. ed. 3, enl. 120 p. 1872. Lockwood. Contains synopsis of principal kinds of foundation works, with the usual modes of treatment. Includes remarks on footings, planking, sand, concrete, pile-driving, caissons, and coffer-dams.

DRAKE, FRANCIS E. Concrete-Pile Holder Foundation. 1915. (*Gas Age*, v. 35, p. 221-224.) Paper before New England Association of Gas Engineers. Gas holder constructed on filled ground. Hardpan was reached by the piles at an average depth of 13 ft. 9 in. The "pedestal pile" was used.

DYE, IRA W. Tower Foundations for the Cristobal-Balboa Transmission Line. 1915. (*Proceedings*, Engrs.' Soc. of Western Pennsylvania, v. 30, p. 973-990.) Describes work under difficult conditions. Some of the foundations were on steep fills, in swamps, and in other unusual locations.

ESPITALLIER, G. La Fondation des Ouvrages en Terrains Vaseux. 1905. (*Le Génie Civil*, v. 47, p. 329-331.) Considers difficult foundation work at Saigon, Indo-China, on muddy ground.

——Abstract. 1906. (*Minutes of Proceedings*, Inst. C. E., v. 164, p. 451-452.)

EWEN, JOHN M. Foundations for Chicago Buildings. 1905. (*Journal*, Western Soc. of Engrs., v. 10, p. 687-704.) Gives information regarding nature of Chicago soils at different depths and discusses foundation practice.

FOUNDATION FOR THE CANAL AND CLAIBORNE ELECTRIC RAILWAY POWER House, New Orleans, La. 1 dr. 1897. (*Engineering News*, v. 38, p. 124-125.) Foundation soil is new material to depth of about 35 ft. underlaid with natural stratum of gravel and shells, from 3 to 7 ft. thick. Building walls were carried on piles.

FOUNDATION WORK ON THE MUNICIPAL BUILDING, NEW YORK. 3 diag. 1910. (*Engineering Record*, v. 62, p. 46-48.) Gives tests of bearing value of sand, p. 46-47.

FOUNDATIONS AND UNDERPINNING IN MUD AND SAND. 1906. (*Engineering Record,* v. 53, p. 167.) Describes foundations of eight-story building in New York City.

FOUNDATIONS FOR HEAVY BUILDINGS. 1891. (*Engineering Record,* v. 23, p. 140-142.) Mostly a discussion of the ever-increasing foundation loads per unit area and of the proper means of securing safety.

FOUNDATIONS FOR THE NEW CITY HALL IN CHICAGO. 2 ill. 1909. (*Engineering Record,* v. 59, p. 745-747.) Bed-rock at site of building is overlaid by successive layers of water-bearing strata, hardpan, blue clay, yellow clay, muck, and refuse filling material.

FOUNDATIONS OF THE FIREMEN'S INSURANCE BUILDING IN NEWARK. 1911. (*Engineering Record,* v. 64, p. 636-637.) Excavations were through quaking sand and a subterranean stream. Steel sheet-piling was used.

FOUNDATIONS OF THE MUNICIPAL BUILDING, NEW YORK CITY. 2 diag., 3 dr., 6 ill. 1910. (*Engineering News,* v. 64, p. 523-529.) Partly on rock and partly on sand. Results of tests are given.

FOUNDATIONS OF THE NEW POST OFFICE AND GOVERNMENT BUILDING AT Chicago. 5 diag., 1 dr. 1898. (*Engineering News,* v. 39, p. 66-68.) Indicates successive soil strata for four borings, showing, in general, blue clay of varying consistency between layer of filling and the hardpan.

FOUNDATIONS OF THE NORTHWESTERN RAILWAY TERMINAL, CHICAGO. 1909. (*Engineering Record,* v. 59, p. 595-597.) Details of unusual foundation work through soft soil, blue clay, and hardpan to rock.

FOWLER, CHARLES EVAN. Practical Treatise on Subaqueous Foundations, Including the Coffer-dam Process for Piers and Dredges and Dredging, with Numerous Practical Examples from Actual Work. ed. 3, rev. and enl. 814 p. 1914. John Wiley & Sons, New York. [Bearing value of soils], p. 409-423, 491.

FRYE, ALBERT I. Civil Engineers' Pocket-book. 1611 p. 1913. Van Nostrand. Includes data on theory and properties of earth, earthworks, foundations, retaining walls, piles and pile driving, earth dams. Bibliographical notes at end of each main division.

GAUDARD, JULES. On Foundations. 1877. (*Minutes of Proceedings,* Inst. C. E., v. 50, p. 112-147.) Gives various methods of foundation construction, safe loads for various soils, and information on friction of cylinders in soils.

GOTTLIEB, A. Foundations and Floors for the World's Fair Buildings. 1891. (*Engineering Record,* v. 25, p. 15-16.) After tests with loaded platforms, spread foundations were adopted on a sandy bottom with clay substrata.

GREAT BRITAIN—PATENT OFFICE. Abridgment of Specifications: Class 68, Hydraulic Engineering. 1855-1908. Classified abstracts of British patents. Includes subaqueous structures, caissons, coffer-dams, piles and pile-driving, sea walls, and retaining walls.

GRUEBER, PAUL. Wetzmann Retaining Dam. 7 diag. 1886. (*Engineering News,* v. 15, p. 353-354.) Translated from *Central-Blatt der Bauverwaltung,* June, 1885. The principles of the erosive action of torrential streams, as developed by Alexander Surrell, were successfully applied to protective works in the Alps.

GUARINI, EMILE. Mechanical Compression of the Ground in the Construction of Foundations. 1906. (*Scientific American,* v. 108, n. s., v. 94, p. 476-477.) Method "consists in compressing the ground by treating it with concrete in such a way as to compress it laterally and depthwise."

HASELER, E., and others. Erd- und Felsarbeiten. Erdrutschungen. Stütz- und Futtermauern. 414 p. 12 folding pl. 1905. (Handbuch der Ingenieurwissenschaften, pt. 1, v. 2, ed. 4, rev.) Literatur, p. 403-407.

HATCH, JAMES N. Chicago Foundations. 1 diag., 3 dr. 1904. (*Architects' and Builders' Magazine,* v. 5, p. 555-561; v. 6, p. 18-24.) Depth and nature of strata determined from borings at time of construction of Post Office Building. Discusses generally soils underlying business district of Chicago.

HICKS, LEWIS A. Standardized Square Footings of Reinforced Concrete. 62 p. 1915. Accompanied by a working chart, 15⅞ by 17⅞ in. Presentation of methods for practical utilization of the data given in *Bulletin* 67 of the University of Illinois.

HILL, GEORGE. Foundations. 1893. (*American Architect,* v. 41, p. 179-182.) Discusses testing of soils, bearing value of various soils, bearing value of piles, and treatment of soils.

HOWE, MALVERD A. Foundations: a Short Text-book on Ordinary Foundations, Including a Brief Description of the Methods Used for Difficult Foundations. 110 p. 1914. John Wiley and Sons, New York. "References" at the end of each chapter. "Supporting Capacity of Soils," p. 1-12.

HUNT, RANDELL. Principles Governing the Design of Foundations for Tall Buildings. 1896. (*Journal,* Assoc. Eng. Soc., v. 17, p. 1-26.) Gives brief information and suggestions regarding soil foundations and the desirable factor of safety in loading soils.

HUTTON, W. R. Foundations of High Buildings. 1893. (*Engineering Record,* v. 28, p. 268-270.) Paper before Congress of Architects, Chicago. Discusses New York and Chicago foundation practice, and the nature of the underlying soils. Considers extent of lateral displacement of soil due to vertical load.

JACOBY, HENRY S., and DAVIS, ROLAND P. Foundations of Bridges and Buildings. 603 p. 1914. McGraw-Hill Book Company, New York. Excellent book, covering subject thoroughly. "Bearing Power of Piles," p. 75-115. Formulas for bearing power of concrete piles, p. 161-162. "Tests fór Bearing Capacity," p. 531-534. "Values of Bearing Capacity," p. 534-537. "References to Engineering Literature," p. 562-597.

KENT, WILLIAM. Mechanical Engineer's Pocket-book. ed. 9. 1526 p. 1916. Wiley. Brief treatment of properties of soils, foundations, piles, etc.

KIDDER, FRANK EUGENE. Architects' and Builders' Pocket-book. ed. 16. Thomas Nolan, Editor-in-Chief. 1816 p. 1916. Wiley. Includes considerable data on soil characteristics as relating to bearing power, foundations, piles, retaining walls, etc.

KOENIG, ARNOLD C. Dams on Sand Foundations; Some Principles Involved in Their Design, and the Law Governing the Depth of Penetration Required for Sheet-piling. 4 dr. 1911. (*Transactions,* Am. Soc. C. E., v. 73, p. 175-189.) Discussion, 2 diag., 6 dr., p. 190-224.

KURDJUMOFF, V. J. Foundations on Natural Ground. 1893. (*Minutes of Proceedings,* Inst. C. E., v. 111, p. 414-415.) Abstract from *Der Civilingenieur,* 1892, p. 293. Deals with foundations in loose or sandy soils. Uses photographic methods. Finds that Rankine's formula, based on the theory of granular masses, does not give accurate results.

LEA, SAMUEL H. Foundation Construction for the New State Capitol of South Dakota. 2 dr., 1 ill. 1908. (*Engineering Record,* v. 57, p. 437-438.) Tests of bearing capacity of hard, firm soil a few feet below surface.

LEONARD, HUGH. Foundations for Buildings in Calcutta. 4 dr. 1895. (*Journal,* Royal Inst. of British Architects, v. 44, p. 593-595.) Results of tests of sandy clay.

——1875. (*Engineering,* v. 20, p. 103.)

——Abstract. 4 dr. 1895. (*Engineering Record,* v. 32, p. 317.)

LESLIE, BRADFORD. Account of the Bridge Over the Gorai River on the Goalundo Extension of the Eastern Bengal Railway. 1872. (*Minutes of Proceedings,* Inst. C. E., v. 34, p. 1-42.) Considers foundations.

LEUREY, L. F. Soft-ground Foundations, Panama-Pacific International Exposition; Penetration and Load Tests of Piles. 1914. (*Engineering News,* v. 72, p. 250-254.) Soil consisted of soft sand and mud (hydraulic-dredge fill). Pile foundations were adopted for a large part of the building area. Test borings were made.

McALPINE, WILLIAM JARVIS. Foundations of the New Capitol at Albany, N. Y. 1874. (*Transactions,* Am. Soc. C. E., v. 2, p. 287-288.) Briefly discusses bearing value of soils, and gives results of tests.

McALPINE, WILLIAM JARVIS. Foundations of the New Capitol at Albany, New York. 1879. (*Minutes of Proceedings,* Inst. C. E., v. 57, p. 198-207.) Describes experiments, and also various considerations taken into account for determining the bearing value of soil.

MAHAN, D. H. Treatise on Civil Engineering; Revised and Edited by De Volson Wood. 637 p. 1893. Includes treatment of "Foundations of Structures on Land," and of retaining walls.

MARKS, LIONEL SIMEON, and others, *ed.* Mechanical Engineers' Handbook. 1836 p. 1916. McGraw. Data on bearing value of soils, pile foundations, safe loads for piles, and retaining walls, p. 1264-1268.

MAYER, RUDOLF. Ueber die Bedingungen einer Gleichförmigen Druckvertheilung in den Fundamenten. 3 diag. 1897. (*Zeitschrift,* Oesterreichischen-Ingenieur-und Architekten-Vereines, v. 49, p. 34-36.)

MERRIMAN, MANSFIELD. American Civil Engineers' Pocket Book. 1380 p. 1911. Wiley. Data on foundations, bearing capacities of soils, retaining walls, earth pressure, etc.

METHODS USED TO PROTECT THE FOUNDATION OF A HEAVY TOWER DURING Adjacent Subway Construction in Boston, Mass. 1914. (*Engineering-Contracting*, v. 42, p. 604-605.) Means for protecting foundation of Old South Church during subway construction. Steel sheet-piling driven on both sides of the subway, and the soil inside the coffer-dam thus formed grouted. The subway was then constructed in short sections, each section being completed before excavation for the next section had reached a depth of more than 10 ft.

MOLESWORTH, GUILFORD L., and MOLESWORTH, HENRY B. Pocket-book of Useful Formulæ and Memoranda for Civil, Mechanical and Electrical Engineers. ed. 27. 936 p. 1913. Includes data on retaining walls, foundations, etc.

O'ROURKE, JOHN F. Foundations for Heavy Structures in Deep Alluvial Soil. 1907. (*Engineering News*, v. 58, p. 87-88.) Much information on bearing value of such soils.

PATTON, WILLIAM MACFARLAND. Practical Treatise on Foundations, Explaining Fully the Principles Involved, Supplemented by Articles on the Use of Concrete in Foundations. ed. 2, enl. 549 p. 1906. John Wiley and Sons, New York. [Bearing resistances of materials] p. 3-8, 146, 149, 213-216, 291-294, 308, 343-352, 358, 532. [Bearing power of piles and formulas], p. 376-378, 395, 472.

PATTON, WILLIAM MACFARLAND. Treatise on Civil Engineering. ed. 2. 1654 p. 1907. Includes articles on foundations and foundation-beds, ordinary earthwork, retaining walls, reservoir walls, dams and weirs, etc.

POWELL, GEORGE T. Foundations and Foundation Walls for All Classes of Buildings, Pile Driving, Building Stones and Bricks, Pier and Wall Construction, Mortars, Limes, Cements, Concretes, Stuccos, etc., to Which is Added a Treatise on Foundations, with Practical Illustrations of the Method of Isolated Piers, as Followed in Chicago, by Frederick Bauman. ed. 4. 166 p. 1889. W. T. Comstock.

PRELIMINARY FOUNDATION TESTS FOR THE ST. PAUL BUILDING. 1896. (*Engineering Record*, v. 33, p. 388.) Loading tests on sand soil with solid rock about 86 ft. below the curb.

RANKINE, WILLIAM JOHN MACQUORN. Manual of Civil Engineering. ed. 24. 822 p. 1911. Includes considerable information on soils for foundation work. "Strength and Stability of Earthwork in General," p. 315; "Ordinary Foundations," p. 377; "Timber, Iron and Submerged Foundations," p. 601.

REINFORCED CONCRETE FOOTINGS OF SPECIAL CONSTRUCTION. 1 dr. 1915. (*Engineering Record*, v. 72, p. 355-356.) Outside corner columns for the ten-story storehouse of the Appleton Company at Lowell, Mass., supported on low-bearing soil, without extending beyond the lot line.

REINFORCEMENT OF ENGINE FOUNDATIONS ON SOFT SOIL. 1907 (*Engineering Record*, v. 55, p. 254.) Soil consisted of sand and silt. Foundations of plant were carried down to clean building sand, on which footings were spread. Foundations of engines were extended down to lower stratum.

REITER, J. Pneumatic Caisson Foundations of the Prague-Smichow Bridge. 1878. (*Minutes of Proceedings*,, Inst. C. E., v. 51, p. 288-290.) Abstract from *Mittheilungen* des Architekten- und Ingenieur-Vereines im Königreiche Böhmen, v. 11, p. 7-9, 13-16; v. 12, p. 20-24. Treats of frictional resistance of a caisson in sand or gravel, p. 290.

RIPLEY, CHARLES M. Concrete Foundations of the Tallest Skyscraper. 4 dr., 1 ill. 1907. (*Cement*, v. 8, p. 52-59.) Considers soils underlying Singer Building, New York City, and information gained by borings to level of solid rock.

RIPLEY, CHARLES M. Foundation Problems in New York City; How the Collapse of Neighboring Buildings is Prevented and Caissons are Sunk to Bedrock, Using the Moran Air Lock. 1907. (*Iron Age*, v. 79, p. 480-484.)

——1907. (*Compressed Air*, v. 12, p. 4381-4388.)

ROHAN, W. D. Grouting Natural Soils for Bridge and Building Foundations. 1910. (*Engineering-Contracting*, v. 34, p. 545.) Favors use of grout in foundation work.

ROYAL BANK BUILDING FOUNDATION WORK. 1914. (*Canadian Engineer*, v. 26, p. 525-529.) Describes foundations of twenty-story building. Soil is composed of an upper layer of clay extending to a depth of about 30 ft., with a deposit of moderately hard and shaly clay beneath for a depth of from 2 to 3 ft. Below is a very hard, dense shale, with little variation in a depth of 600 ft.

SCHAEFER, R. Unterdruck bei Staumauern. 10 diag., 1 ill., 2 pl. 1913. (*Zeitschrift für Bauwesen,* v. 63, p. 101-118.)

SCHILL, A. K. J. Calculation of Reinforced Concrete Foundation Slabs. 1911. (*Minutes of Proceedings,* Inst. C. E., v. 186, p. 479.) Abstract from *De Ingenieur*, 1911, v. 26, p. 329-330.

SCHMITT, EDUARD. Fundamente. 1901. (Handbuch der Architektur, v. 3, pt. 1, p. 281-406.) Contains many bibliographies. Includes section on character and investigation of the soil on the sites of structures, and precautions against sinking and sliding. Also includes a section on the construction of masonry, concrete, concrete and sand, and grillage foundations. A third section takes up deep and submerged foundations.

SCHUYLER, JAMES DIX. Reservoirs for Irrigation, Water-Power and Domestic Water-Supply, with an Account of Various Types of Dams and the Methods, Plans and Cost of Their Construction. ed. 2. 556 p. 1908. Contains chapter on "Earthen Dams," p. 416-451.

SICCAMA, H. T. H. Foundations. 1903. (*Bulletin News,* v. 85, p. 646-647.) Paper before Civil and Mechanical Engineers' Society. Shows effect of nature of soil on depth and construction of foundations.

——1903. (*American Architect,* v. 82, p. 92-93.)

SKINNER, FRANK W. Development of Building Foundations. 7 ill., 12 dr. 1908. (*Engineering Record,* v. 57, p. 412-422.) Gives tests of bearing value of soil and piles, p. 413.

SPREAD FOUNDATIONS IN HAMBURG. 1911. (*Engineering Record,* v. 63, p. 659.) Editorial, describing foundations of very large bearing surface.

STREET, WILLIAM C. Foundations. 1882. (*Van Nostrand's Engineering Magazine,* v. 26, p. 337-342.) Paper read before the Civil and Mechanical Engineers' Society. Gives experience with foundations on sand and clay.

STRESS, WILLIAM H. Method for Figuring Foundation Pressures Under U-Abutments. 3 dr. 1910. (*Engineering Record,* v. 62, p. 560.)

TALBOT, ARTHUR N. Reinforced Concrete Wall Footings and Column Footings. 114 p. 1913. (University of Illinois Engineering Experiment Station, *Bulletin* 67.) Extensive tests. Bearing value of soils is discussed briefly on p. 7. *See also* Hicks, Lewis A.

TALL BUILDING FOUNDATIONS ON SOFT CLAY. 1907. (*Engineering Record,* v. 55, p. 731-732.) Tests of bearing capacity of clay in connection with construction of 15-story building in Toronto.

TESTING FOUNDATIONS AT THE MUNICIPAL BUILDING, NEW YORK. 3 dr., 1 ill. 1911. (*Engineering Record,* v. 63, p. 196-197.) Gives results of loading tests, with pier resting on sand, about 35 ft. below the surface and 80 ft. above rock level.

TRAUTWINE, JOHN C. Civil Engineer's Pocket-book. ed. 18. 1079 p. 1907. Includes sections on Foundations (p. 582-600), and Retaining Walls (p. 603-613).

USINA, D. A. Recent Developments in Pneumatic Foundations for Buildings. 17 dr. 1908. (*Transactions,* Am. Soc. C. E., v. 61, p. 211-221.) Discussion, 3 ill., 4 dr., p. 222-237. Reviews development of foundations in New York and refers to pneumatic foundations as most economical type for maximum bearing pressure.

——Abstract. 1908. (*Minutes of Proceedings,* Inst. C. E., v. 173, p. 357.)

VERNON-HARCOURT, LEVESON FRANCIS. Civil Engineering as Applied in Construction. 624 p. 1902. Includes brief treatment of foundations, and of earthwork in cuttings and embankments.

WADDELL, J. A. L. Bridge Engineering. 2 v. 2177 p. 1916. Wiley. An important work. Contains chapter on "Foundations in General," v. 1, p. 946-972, giving considerable data on permissible pressures. Also contains a chapter on "Piles and Pile Driving," v. 1, p. 1008-1019, in which is included material on characteristics, compacting, and equilibrium of soils.

WEGMANN, EDWARD. Design and Construction of Dams, Including Masonry, Earth, Rock-fill, Timber, and Steel Structures; also Principal Types of Movable Dams. ed. 6. 529 p. 1911. "Bibliography," p. 507-519. Includes chapter on "Earthen Dams," p. 221-247.

YOUNG, EVELYN HENRY. Foundations in "Black Cotton Soil" in India. 1893. (*Minutes of Proceedings,* Inst. C. E., v. 113, p. 323-326.) Soil is rich, black earth composed of disintegrated trap, which swells and exerts an enormous pressure, when filled with water.

RETAINING WALLS, INCLUDING LATERAL EARTH PRESSURE.

See also **Granular Materials.**

ALEXANDER, T., and THOMSON, A. W. Elementary Applied Mechanics. 575 p. 1902. Contains chapters, "Application of the Ellipse of Stress to the Stability of Earthwork," p. 70-86, and "The Scientific Design of Masonry Retaining Walls."

ALLEN, J. ROMILLY. Investigation of the Question of the Thrust of Earth Behind a Retaining Wall. 3 diag. 1877. (*Van Nostrand's Engineering Magazine,* v. 17, p. 155-158.) Mathematical solution.

ALLEN, KENNETH. Design of Retaining Walls. 1892. (*Engineering Record,* v. 26, p. 341-342, 356-357, 374, 393.) On practical design of retaining walls, sea walls, and dock walls. Illustrated with actual designs.

AMERICAN RAILWAY ENGINEERING AND MAINTENANCE OF WAY ASSOCIAtion. [Report of Committee on] Retaining Walls and Abutments. 1909. (*Proceedings,* Tenth Annual Convention, Am. Ry. Eng. and Maintenance of Way Assoc., p. 1317-1337.) Gives information showing practice of various railroads in the designing of retaining walls. Committee submits method of determining earth pressures based on Rankine's formula.

——Condensed. 1909. (*Engineering Record,* v. 60, p. 288-290.)

AUDE. Nouvelles Expériences sur la Poussée des Terres. 1849. (*Comptes Rendus Hebdomadaires* des Séances de l'Académie des Sciences, v. 28, p. 565-566.) Short review of Audé's work presented by Poncelet.

BAKER, BENJAMIN. Actual Lateral Pressure of Earthwork. 1881. (*Minutes of Proceedings,* Inst. C. E., v. 65, p. 140-186.) Discussion, p. 187-241. Aims to present data on actual lateral pressure of earthwork, as distinguished from "text-book" pressures, which latter the author holds to be generally incorrect.

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CLAVENAD. Mémoire sur la Stabilité, les Mouvements, la Rupture des Massifs en Général, Cohérents ou sans Cohésion. Quelques Considérations sur la Poussée des Terres, Etude Spéciale des Murs de Soutènement et de Barrages. 64 diag. 1887. (*Annales des Ponts et Chaussées,* ser. 6, v. 13, p. 593-683.)

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CORNISH, L. D. Fallacies in Retaining Wall Design and the Lateral Pressure of Saturated Earth. 1916. (United States Corps of Engineers, *Professional Memoirs*, v. 8, p. 161-172.) Discussion, p. 173-195. Treats of lateral pressure of saturated soils in connection with the design of retaining walls. Presents considerable mathematical data on the treatment of saturated soil in such design work.

COUPLET. De la Poussée des Terres Contre leurs Revestemens et la Force des Revestemens qu'on Leur Doit Opposer. 8 pl. 1726-1728. (*Histoire* de l'Académie Royale des Sciences, v. 28, p. 106-164; v. 29, p. 132-141; v. 30, p. 113-138.)

COUSINERY. Détermination Graphique de l'Epaisseur des Murs de Soutènement. 1 pl. 1841. (*Annales des Ponts et Chaussées*, ser. 2, v. 2, p. 167-184.) Develops a method of graphical determination of thickness of retaining walls. Shows how to apply the theory of earth pressure in connection with this graphical construction.

CRAMER, E. Die Gleitfläche des Erddruck-prismas und der Erddruck gegen geneigte Stützwände. 4 diag. 1879. (*Zeitschrift für Bauwesen*, v. 29, p. 521-526.)

CRELLE. Zur Statik unfester Körper. An dem Beispiele des Drucks der Erde auf Futtermauern. 1 pl. 1850. (*Abhandlungen* der Königlichen Akademie der Wissenschaften zu Berlin, v. 34, p. 61-97.) To be found in section "Mathematische Abhandlungen."

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CURIE, J. Note sur la Brochure de M. Benjamin Baker Intitulée: "The Actual Lateral Pressure of Earthwork." 9 diag. 1882. (*Annales des Ponts et Chaussées*, ser. 6, v. 3, p. 558-592.) Criticism of Baker's paper in *Minutes of Proceedings*, Inst. C. E., v. 65, p. 140.

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CURIE, J. Sur la Théorie de la Poussée des Terres. 1 diag. 1873. (*Comptes Rendus Hebdomadaires* des Séances de l'Académie des Sciences, v. 77, p. 778-781.) Reply to Saint-Venant's criticism in same volume.

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PONCELET. Mémoire sur la Stabilité des Revêtements et de leurs Fondations. 1840. (*Comptes Rendus Hebdomadaires* des Séances de l'Académie des Sciences, v. 11, p. 134-140.) Review of the author's 270-page essay published in *Mémorial de l'Officier du Génie, No. 13.* Author is an able supporter of Coulomb's theory.

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RESAL, JEAN. Poussée des Terres. 2 v. 1903-1910. (Enzyklopädie des Travaux Publics.) v. 1. Stabilité des Murs de Soutènement. v. 2. Théorie des Terres Cohérentes.—Applications.—Tables Numériques. Purely theoretical work on earth pressures as affecting the design of structures. v. 1 deals entirely with soils lacking cohesion.

REUTERDAHL, ARVID. From the Soil Up: A New Method of Designing. 1914. (*Engineering-Contracting,* v. 42, p. 581-585.) Considers especially retaining wall design. Advocates starting with the bearing capacity of the soil, and working from that basis.

ROSE, W. H. Formulas for the Design of Gravity Retaining Walls. 1910. (*Engineering-Contracting,* v. 34, p. 115-117.) From *Professional Memoirs,* Corps of Engineers, U. S. Army.

SAINT-VENANT, de. Examen d'un Essai de Théorie de la Poussée des Terres Contre les Murs Destinés à les Soutenir. 1873. (*Comptes Rendus Hebdomadaires* des Séances de l'Académie des Sciences, v. 73, p. 234-241.) Criticizes Curie's theory, and defends the so-called rational theory developed by Levy.

SAINT-VENANT, de. Poussée des Terres. Comparaison de ses Evaluations au Moyen de la Considération Rationnelle de l'Equilibre-limite, et au Moyen de l'Emploi du Principe dit de Moindre Résistance, de Moseley. 1870. (*Comptes Rendus Hebdomadaires* des Séances de l'Académie des Sciences, v. 70, p. 894-897.)

SAINT-VENANT, de. Rapport sur un Mémoire de M. Maurice Levy, Présenté le 3 Juin, 1867, Reproduit le 21 Juin, 1869, et Intitulé: Essai sur une Théorie Rationnelle d'Equilibre des Terres Fraîchements Remuées, et ses Applications au Calcul de la Stabilité des Murs de Soutènement. 1870. (*Comptes Rendus Hebdomadaires* des Séances de l'Académie des Sciences, v. 70, p. 217-235.) Report of a committee, giving a historical review of the works on earth pressure, and discussing in detail Maurice Levy's theory.

SAINT-VENANT, de. Recherche d'une Deuxième Approximation dans le Calcul Rationnel de la Poussée Exercée, Contre un Mur dont la Face Postérieure a une Inclinaison quelconque, par des Terres non Cohérentes dont la Surface Supérieure s'Elève en un Talus Plan quelconque à Partir du Haut de Cette Face du Mur. 1 diag. 1870. (*Comptes Rendus Hebdomadaires* des Séances de l'Académie des Sciences, v. 70, p. 717-724.) Based on Levy's theory.

SAINT-VENANT, de. Sur une Détermination Rationnelle, par Approximation, de la Poussée qu' Exercent des Terres Depourvues de Cohesion, Contre un Mur ayant une Inclinaison quelconque. 3 diag. 1870. (*Comptes Rendus Hebdomadaires* des Séances de l'Académie des Sciences, v. 70, p. 229-235, 281-286.) Development of Levy's theory.

SAINT-VENANT, de. Sur une Evaluation, ou Exacte ou d'une Très Grande Approximation, de la Poussée des Terres Sablonneuses Contre un Mur Destiné à les Soutenir. 1884. (*Comptes Rendus Hebdomadaires* des Séances de l'Académie des Sciences, v. 98, p. 850-852.) Based on Boussinesq's works.

SCHAFFER. Erddruck und Stützwände. 1 diag., 1 pl. 1878. (*Zeitschrift für Bauwesen,* v. 28, p. 527-548.)

SCHMITT, EDUARD. Empirische Formeln zur Bestimmung der Stärke der Futtermauern. 1871. (*Zeitschrift,* Oesterreichischen Ingenieur-und Architekten-Vereines, v. 23, p. 336-338.) Mathematical calculations on the basis of Rebhann's tables.

SCHWEDLER, J. W. [Unterschnittene Futtermauern.] 1871. (*Zeitschrift für Bauwesen,* v. 21, p. 280-282.) Discussion of the formula derived by Schwedler at a meeting of the Architekten-Verein zu Berlin.

SERBER, D. C. Stability of Sea Walls. 15 diag. 1906. (*Engineering News,* v. 56, p. 198-200.) Gives method of design.

——Brief abstract. 1906. (*Le Génie Civil,* v. 50, p. 32.)

SHEET-PILING AND EARTH PRESSURE. 1907. (*Engineering Record,* v. 56, p. 528.) Refers particularly to paper on "The Bracing of Trenches and Tunnels," by J. C. Meem.

SIEGLER. Expériences Nouvelles sur la Poussée du Sable. 1887. (*Annales des Ponts et Chaussées,* ser. 6, 13, p. 488-505.) Experimental method for studying reactions between masses of earth and their supporting walls. Friction dynamometer was used to determine intensity of pressure.

——Condensed translation. "New Experiments on the Thrust of Sand." 1887. (*Scientific American Supplement,* v. 24, p. 9724-9725.)

SINGER, MAX. Fliessende Hänge. 1902. (*Zeitschrift,* Oesterreichischen Ingenieur- und Architekten-Vereines, v. 54, pt. 1, p. 190-196.) Describes yielding of sides of railway cutting in valley of the Eger, Austria, with methods used for retaining embankment.

SINKS, F. F. Analysis and Design of a Reinforced Concrete Retaining Wall. 1905. (*Engineering News,* v. 53, p. 8-9.)

SINKS, F. F. Design for Reinforced Concrete Retaining Wall. 1904. (*Railroad Gazette,* v. 37, p. 676-677.) Letter.

SKIBINSKI, CARL. Ueber Stützmauerquerschnitte. 1898. (*Zeitschrift,* Oesterreichischen Ingenieur- und Architekten-Vereines, v. 45, p. 666-670.)

SKIBINSKI, KARL. Theorie des Erddrucks auf Grund der neueren Versuchen. 1 diag., 1 pl. 1885. (*Zeitschrift,* Oesterreichischen Ingenieur- und Architekten-Vereines, v. 37, p. 65-77.) Develops his own theory of earth pressure based on the experimental work of Forchheimer, Gobin, and Darwin. Gives a graphical construction of his theory, and methods of practical application.

SPILLNER, E. Stützmauern. 1904. (Handbuch der Architektur, ed. 3. v. 3, pt. 6, p. 182-197.) "Literatur," p. 196.

STRUKEL, M. Beitrag zur Kenntniss des Erddruckes. 2 diag., 4 dr. 1888. (*Zeitschrift,* Oesterreichischen Ingenieur- und Architekten-Vereines, v. 40, p. 119-125.) Critical review of the salient points of the earth pressure theory as developed by Coulomb, Rebhann, and others. In support of his own views, gives results of some experiments.

SYLVESTER, J. J. On the Pressure of Earth on Revetment Walls. 1 diag. 1860. (*London, Edinburgh and Dublin Philosophical Magazine and Journal of Science,* ser. 4, v. 20, p. 489-499.) Criticism of theories of Coulomb and Rankine.

TATE, JAMES S. Surcharged and Different Forms of Retaining Walls. 59 p. 1873. Van Nostrand. Theoretical calculations for retaining walls.

——1873. (*Van Nostrand's Engineering Magazine,* v. 9, p. 481-494.)

THORNTON, WILLIAM M. Retaining Walls. 7 diag. 1879. (*Van Nostrand's Engineering Magazine,* v. 20, p. 313-318.) Concise and simplified account of the theory of earth pressure and its application to the design of retaining walls.

VAN BUREN, JOHN D., JR. Quay and Other Retaining Walls. 6 diag. 1872. (*Transactions,* Am. Soc. C. E., v. 2, p. 193-221.) Establishes practical formulas for the dimensions of walls of various shapes and under various conditions. Follows Coulomb's theory. An appendix gives a number of mathematical relations.

VEDEL, P. Theory of the Actual Earth Pressure and Its Application to Four Particular Cases. 1894. (*Journal,* Franklin Inst., v. 138, p. 139-148, 189-198.) Mathematical calculation.

WALMISLEY, A. T. Retaining Walls. 1907. (*The Builder,* London, v. 93, p. 647-648.) Discusses calculations of earth pressure, foundations, etc.

WEINGARTEN. [Die Theorie des Erddrucks.] 1 diag. 1870. (*Zeitschrift für Bauwesen,* v. 20, p. 122-124.) Abstract of a paper read before the Architekten-Verein zu Berlin.

WESTON, W. E. Tables for Use in Determining Earth Pressure on Retaining Walls. 1911. (*Engineering News,* v. 65, p. 756-757.)

WINKLER, E. Neue Theorie des Erddruckes. 19 diag. 1871. (*Zeitschrift,* Oesterreichischen Ingenieur- und Architekten-Vereines, v. 23, p. 79-89, 117-122.)

WOODBURY, D. P. On the Horizontal Thrust of Embankments. 1 diag. 1859. (*Mathematical Monthly,* v. 1, p. 175-177.) Mathematical paper.

WOODBURY, D. P. Remarks on Barlow's Investigation of "the Pressure of Banks, and Dimensions of Revetments." 2 diag. 1845. (*Journal,* Franklin Inst., v. 40, p. 1-7.)

PILES.

GENERAL.

See also **Foundations.**

ABBOTT, HUNLEY. Discussion of the Carrying Capacity of Bulb-pointed Concrete Piles. 1911. (*Engineering-Contracting,* v. 35, p. 41-43.) Takes into consideration different kinds of soils and "safe bearing power of soils."

BENABENQ, J. Résistance des Pieux : Théorie et Applications. 16 diag., 29 dr. 1911. (*Annales des Ponts et Chaussées,* ser. 9, v. 5, p. 263-355 ; ser. 9, v. 6, p. 475-543.) Deals with the bearing value of piles from a mathematical standpoint. The first part treats the subject from a new standpoint, viz., that of static equilibrium. The second part follows more or less normal lines. Numerous formulas are deduced. Gives many tables of constants.

——Abstract. 1912. (*Minutes of Proceedings,* Inst. C. E., v. 158, p. 475-476.)

——Abstract. 1912. (*Engineering Record,* v. 65, p. 248.)

——Condensed. 1912. (*Le Génie Civil,* v. 60, p. 246-250.)

CEZANNE. Notice sur le Pont de la Theiss et sur les Fondations Tubulaires. 1859. (*Annales des Ponts et Chaussées*, ser. 3, v. 17, p. 334-382.) Loading of piles at Szegedin, p. 340-341.

COLUMN ACTION IN PILES. 1908. (*Engineering News*, v. 60, p. 18-19.) Under the conditions of river pier construction, also in other cases, the free length of pile is great. The vertical strength of such columns is very much below the bearing value that could be developed in a firm gravel stratum. To design long soft-ground marine piling for pure bearing value incurs the risk of overloading the piles as columns.

CONCRETE PILE WALL FOUNDATIONS. 1 dr., 1 ill. 1904. (*Engineering Record*, v. 50, p. 431-432.) Describes foundation work for the United States Express Company's building, New York. Gives results of pile tests.

COTHRAN, THOMAS W. Form for Pile-driving Records Used on the Norfolk and Southern Railway. 1 chart, 1 dr. 1907. (*Engineering News*, v. 57, p. 596.) Letter to editor.

ECCENTRIC LOAD ON PILES OR RIVETS. 1912. (*Engineering News*, v. 67, p. 1190.) Note giving simple formula for section modulus of eccentrically loaded piles or rivets.

FARGO, WILLIAM G. Experience with Steel Sheet-piling in Hard Soils. 1907. (*Engineering News*, v. 57, p. 374-375.) Paper before Michigan Engineering Society. Methods and costs. Clay hardpan was encountered.

——1907. (*Engineering-Contracting*, v. 27, p. 193-195.)

——Condensed. 1907. (*Engineering Record*, v. 55, p. 175-176.)

GOODRICH, E. P. Column Action in Piles; Stiffening Piles by Rip-rap. 1908. (*Engineering News*, v. 60, p. 41.) Letter to editor.

GOODRICH, E. P. Why Not a Rational Specification for a Wooden Pile? 1915. (*Engineering Record*, v. 71, p. 627-628.) Letter to editor.

HOOPER, HENRY. Description of the Pier at Southport, Lancashire. 1861. (*Minutes of Proceedings*, Inst. C. E., v. 20, p. 292-299.) Gives load used on cast-iron piles, p. 295.

HOWE, W. B. W. Some Facts of Experience in Pile Driving. 1892. (*Engineering News*, v. 28, p. 543-545.) Letter to editor and reply by editor, A. M. Wellington.

HURTZIG, ARTHUR CAMERON. Note on the Friction of Timber Piles in Clay. 1 dr., 1 pl. 1881. (*Minutes of Proceedings*, Inst. C. E., v. 64, p. 311-315.) Gives results of observations on frictional resistance to motion of piles and deduces a formula for maximum load.

——Condensed. 1 diag. 1881. (*Van Nostrand's Engineering Magazine*, v. 25, p. 273-276.)

KNIGHT, G. L. Underpinning with Hollow Piles Driven with Point. 1916. (*Engineering News*, v. 76, p. 982-985.) Two-story building underpinned for unit load of 600 lb. pér sq. ft., and increased to four stories. Special pile-driver drove hollow steel piles 25 ft. with lower ends closed by a special projectile-shaped point.

LENTZ, H. Drawing Piles in Cuxhaven Harbor. 1880. (*Minutes of Proceedings*, Inst. C. E., v. 59, p. 396-397.) Abstract from *Deutsche Bauzeitung*, 1879, p. 340. Discusses difficult operation of drawing piles and gives the pull required in certain special cases.

LESSON IN PILE-DRIVING. 1 dr. 1889. (*Engineering News*, v. 22, p. 368.) Editorial, pointing out the faulty pile work of the South Street Bridge, Philadelphia.

MARQUARDSEN, R. P. V. Pressures on Piles Supporting Masonry. 1915. (*Journal*, Western Soc. of Engrs., v. 20, p. 541-547.)

MAYER, RUDOLF. Ueber die Vertheilung des Pfeilerdruckes in den Fundamenten. 4 diag. 1896. (*Zeitschrift*, Oesterreichischen Ingenieur- und Architekten-Vereines, v. 48, p. 654-656.) Discusses the conditions favorable to a uniformly distributed loading on soil.

METHODS OF CONSTRUCTING REINFORCED CONCRETE PILE BENTS FOR THE Atlantic City Boardwalk. 1909. (*Engineering-Contracting*, v. 31, p. 126-128.) Gives method used in constructing a concrete pile foundation in sea sand.

PILE DRIVING; PRINCIPLES OF PRACTICE. 1911. (*Proceedings*, Am. Ry. Eng. and Maintenance of Way Assoc., v. 12, p. 278-302.) Also gives pile record forms. *See also*, Amendments, p. 306.

PILE DRIVING DESTROYS A TUNNEL BY CLAY PRESSURE. 1 diag., 1 ill. 1915. (*Engineering News*, v. 74, p. 404-405.) An 8-ft. brick tunnel under the Cuyahoga River, Cleveland, Ohio, was completely crushed by lateral pressure and flow of the clay subsoil.

PILES AND PILE DRIVING. 1910. (*Proceedings*, Am. Ry. Eng. and Maintenance of Way Assoc., v. 11, pt. 1, p. 185-216.) Gives pile record forms.

PLACING PILE FOUNDATION PIERS FOR NEW BUILDING BEFORE DEMOLITION of Old. 1914. (*Engineering News*, v. 71, p. 910-912.) New York building.

RENO, J. W. Pile Foundations for Tunnels in Soft Ground. 2 dr. 1907. (*Engineering News*, v. 58, p. 43.) Describes Reno pile-driver; gives pressure piles will sustain in soft ground after standing a few days.

SCHURCH, H. Eine eigenartige Eisenbetonpfahlgründung. 1914. (*Beton und Eisen*, v. 13, p. 373-375; v. 14, p. 11-16.) Describes foundations of a hotel in Triest on floating piles, which have no solid bearing.

SHERTZER, TYRRELL B. Another Form for Pile-driving Records. 1 chart. 1907. (*Engineering News*, v. 58, p. 66.) Letter to editor.

STEEL SHEET-PILING FOR RETAINING EARTH UNDER SPREAD FOOTINGS. 1908. (*Engineering Record*, v. 58, p. 15-16.) Describes methods used under old Custom House, New York City. Soil consisted of mixture of clay and brownish sand containing many small stones and a great quantity of water.

STEEL PILING FOUNDATIONS. 1906. (*Engineering Record*, v. 53, p. 246). Considers soil and load on concrete filling of caissons.

USE OF TIMBER IN ENGINEERING STRUCTURES. 1892. (*Engineering*, London, v. 53, p. 412-413.) Editorial indicating the extensive use of wood piling in United States and discussing some formulas for supporting power of piles used by American engineers.

WEIHE, H., *ed.* Rammen und zugehörige Hulfsmaschinen. 1910. (Handbuch der Ingenieurwissenschaften, v. 4, pt. 1, p. 211-312.) "Literatur," p. 309-312. Good review of the use of piles in foundation work.

WELLINGTON, A. M., *ed.* Piles and Pile Driving. 145 p. 1893. Engineering News Pub. Co. Reprint of articles that appeared previously in *Engineering News*, particularly on the safe loading of piles.

WILLMANN, L. von. Tragfähigkeit eingerammter Pfähle. 1906. (Handbuch der Ingenieurwissenschaften, ed. 4, pt. 1, v. 3, p. 88-93.)

THEORY AND FORMULAS.

BAILLAIRGE, C. Supporting Power of Piles. 1902. (*Engineering Record*, v. 45, p. 183-184.) Letter to editor discussing defects of existing formulas and suggesting a board of inquiry into the subject of pile foundations by American and Canadian Governments.

COLE, HOWARD J. Concrete Piles. 12 ill., 1 dr. 1909. (*Transactions*, Am. Soc. C. E., v. 65, p. 467-487.) Discussion, 3 diag., 4 ill., 5 dr., p. 488-513. Important discussion on concrete piles. Formulas for their bearing value are derived.

——Abstract. 4 diag. 1909. (*Engineering-Contracting*, v. 32, p. 308-310.)

DIAGRAMS FOR SAFE LOADS ON PILES. 1 diag. 1912. (*Engineering-Contracting*, v. 37, p. 94.) Constructed with the use of *Engineering News* formula.

ELLIS, G. W. New Pile Formula Desired. 1905. (*Engineering Record*, v. 52, p. 390.) Letter to editor criticizing accepted formulas for bearing value of piles and urging the Engineering Profession to evolve a new one.

[***ENGINEERING NEWS*** **FORMULA.**] 1906. (*Engineering News*, v. 55, p. 499.) Editorial.

FORMULAE FOR SAFE BEARING LOAD OF PILES. 1889. (*Engineering News*, v. 22, p. 368-369.) Editorial digest.

FORMULAE FOR SAFE LOADS OF BEARING PILES. 1888. (*Engineering News*, v. 20, p. 509-512.) A. M. Wellington, editor, derives a formula which later became known as the "*Engineering News* formula." Compares this formula with others.

GOODRICH, ERNEST P. The Supporting Power of Piles. 18 diag., 3 dr. 1902. (*Transactions*, Am. Soc. C. E., v. 48, p. 180-212.) Discussion, p. 213-219. Author develops a general formula for the bearing value of piles, and compares it with numerous existing formulas.

——Abstract. (*Minutes of Proceedings*, Inst. C. E., v. 149, p. 383-384.) *See also* editorial "Supporting Power of Piles," *Engineering Record*, v. 45, p. 97. *See also* letters to editor. 1 diag. 1902. (*Engineering Record*, v. 45, p. 183-184.)

GOODRICH, ERNEST P. The Supporting Power of Piles. 1910. (*Proceedings*, Am. Ry. Eng. and Maintenance of Way Assoc., v. 11, pt. 1, p. 217-236.) Suggests further experimental work for developing a dependable formula.

——Condensed. 1910. (*Engineering-Contracting*, v. 33, p. 371-373.) *See also* editorial, *Engineering Record*, v. 61, p. 744.

GRIFFITH, JOHN H. Ultimate Load on Pile Foundations, Static Theory. 6 diag. 1910. (*Transactions*, Am. Soc. C. E., v. 70, p. 412-441.) Discussion, p. 442-447. *See also* editorial, *Engineering Record*, v. 61, p. 744. Attempts to construct a formula by statical methods.

HAAGSMA, R. Allowable Load on Piles. 1892. (*Engineering*, London, v. 53, p. 553-554.) Discusses his formula which was originally published in *Tijdskrift* van het Koninklyk Instituut van Ingenieurs, 1886-1887, and which is much used in Holland.

HASWELL, CHARLES H. On Formulas for Pile-Driving. 1893. (*Minutes of Proceedings*, Inst. C. E., v. 115, p. 315-320.) Discusses formulas of Rankine, Sanders, Molesworth, and others, and derives his own formula.

HASWELL, CHARLES H. Pile-Driving Formulas: Their Construction and Factors of Safety. 1899. (*Transactions*, Am. Soc. C. E., v. 42, p. 267-276.) Discussion, 1 diag., p. 277-287. *See also Railroad Gazette*, v. 31, p. 608. Reviews and criticizes the pile formulas most in use, and presents for consideration a new one devised by himself.

HENDRY, M. C. Bearing Power of Piles. 1909. (*Canadian Engineer*, v. 16, p. 485-490.) Discusses formulas and their basis, action of soil, etc.

HERING, RUDOLPH, *comp.* Bearing Piles. (*Engineering News*, v. 5, p. 56, 88, 103-104, 116, 124.) Gives formulas for bearing value, size, and location in any foundation.

K., F. P. Short Formula for Bearing Piles. 1888. (*Engineering News*, v. 20, p. 326.) Letter to editor.

KAFKA, RICHARD. Praktische Anwendungen der Methoden zur Bestimmung der zulässigen Pfahlbelastung. 1909. (*Beton und Eisen*, v. 8, p. 161-164, 196-198, 212-216.) Static, geometrical method is used in calculating.

KNOWLES, A. M. Steam-Hammer Pile Formulas; Suggestions and Queries. 1916. (*Engineering News*, v. 75, pt. 1, p. 372-373.) *See also* Rohde, Charles F.

KREUTER, FRANZ. New Method for Determining the Supporting Power of Piles. 1896. (*Minutes of Proceedings*, Inst. C. E., v. 124, p. 373-377.) Analytical method verified experimentally. *See also* editorial "A New Formula for the Supporting Power of Piles." 1896. (*Engineering Record*, v. 33, p. 343.)

——Abstract. 1896. (*Engineering Record*, v. 33, p. 330.)

——Condensed. 1896. (*Railway Review*, v. 36, p. 262.)

KRIEGSMAN, EUGEN F. Pile-Driver Diagram. 1 diag. 1912. (*Engineering Record*, v. 65, p. 417.) Gives logarithmic diagram for ascertaining bearing value of piles by measuring penetration under drop-hammer.

McALPINE, WILLIAM JARVIS. Supporting Power of Piles; and on the Pneumatic Process for Sinking Iron Columns, as Practised in America. With Discussion. 1 pl., 2 dr. 1868. (*Minutes of Proceedings*, Inst. C. E., v. 27, p. 275-319.) Discusses formulas of Sanders, Molesworth, and Weisbach, and deduces his own theory, based on extensive experiments.

McALPINE, WILLIAM JARVIS. Supporting Power of Piles, Both of Wood and Iron, and the Use of the Latter, Either as Piles or Columns of Support for Foundations. 1 ill. 1868. (*Journal*, Franklin Inst., v. 85, p. 98-110, 170-182.) Gives results of tests with various piles. Discusses several pile formulas, and suggests his own for consideration.

MAYNARD, HENRY NATHAN. New Ross Bridge. 1871. (*Minutes of Proceedings*, Inst. C. E., v. 32, p. 146-191.) Discussion refers to Sander's formula for bearing value of piles, p. 165-167.

MILINOWSKI, ARTHUR S. Diagram for Determining the Safe Load on Piles. 1 diag. 1911. (*Engineering News*, v. 65, p. 139.)

PILE DRIVING FACTORS OF SAFETY. 1889. (*Engineering News*, v. 21, pp. 313-314.) Editorial by A. M. Wellington.

PILE DRIVING FORMULAS. 1899. (*Railroad Gazette*, v. 31, p. 608-609.) Reviews Charles H. Haswell's paper, which appeared in v. 27 of *Transactions*, Am. Soc. C. E.

PILE FOUNDATIONS AND PILE DRIVING FORMULAE. 3 diag. 1882. (*Van Nostrand's Engineering Magazine*, v. 27, p. 22-31.) From a circular of the office of Chief of Engineers. Compares experimental results with twenty-one formulas.

RANDOLPH, RD. Pile-Driving Formulas and Practice. 1882. (*Van Nostrand's Engineering Magazine*, v. 27, p. 298-302.)

ROHDE, CHARLES F. Proposes a New Pile Formula. 1916. (*Engineering News*, v. 75, pt. 1, p. 476.) Letter discussing an article on steam-hammer pile formulas: suggestions and queries. *See also* Knowles, A. M.

SAFE LOAD FOR BEARING PILES. 1892. (*Engineering News*, v. 28, p. 469-470.) Editorial by A. M. Wellington.

SANDERS, JOHN. Rule for Calculating the Weight That can be Safely Trusted upon a Pile Which is Driven for the Foundation of a Heavy Structure. 1851. (*Journal*, Franklin Inst., v. 52, p. 304-305.)

STERN, OTTOKAR. Künstliche Befestigung des Baubodensmittels "schwebender" Pilotage. 1907. (*Beton und Eisen*, v. 6, p. 1-4, 56-57.) Calculates bearing value of concrete piles.

STICKNEY, G. F. Diagrams to Determine the Bearing Power of Piles. 2 diag., 1 dr. 1907. (*Engineering Record*, v. 56, p. 720-721.) Gives also the construction of a pile-driver for testing purposes.

STRENGTH OF SHEET-PILING. 1911. (*Proceedings*, Am. Ry. Eng. and Maintenance of Way Assoc., v. 12, pt. 1, p. 303-304.) Derives formulas for pressure due to water, due to dry sand and due to wet slipping material.

SUPPORTING POWER OF PILES. 1902. (*Engineering Record*, v. 45, p. 97. Editorial, pointing out uncertainty of knowledge on the subject. *See also* letter by C. Baillairge, p. 183.

TESTING.

See also **Chemical and Physical Properties of Soils, Testing.**

ACTUAL RESISTANCE OF BEARING PILES. 1893. (*Engineering News*, v. 29, p. 171-173.) Records, with notes, seventeen different examples of actual weight supported by piles at different places.

ADHESION OF TIMBER PILES TO CONCRETE. 6 dr. 1904. (*The Engineer*, London, v. 98, p. 167-168.) Abstract of paper published in "Travaux Publics de Belgique." Gives results of experiments with piles sliding in a mass of concrete and on resistance developed by piles when passing through a concrete platform.

——Condensed. 1904. (*Engineering Record*, v. 50, p. 358-359.)

ALLEYNE, JOHN GAY NEWTON. Dordrecht Railway Bridge, and the Foundations of the Railway Bridge at Rotterdam. 1875. (*Minutes of Proceedings*, Inst. C. E., v. 42, p. 213-227.) Gives tests of piles and formulas used by Dutch engineers for bearing value, p. 216.

BEARING POWER OF PILES. 1894. (*Engineering News*, v. 31, p. 283-284.) Tests made while driving piles for foundations of Chicago Public Library. Bearing value of soils is considered, and formulas tested for bearing power of piles.

BIHLER, C. S. Strength of Piles. 1 diag. 1904. (*Railway and Engineering Review*, v. 44, p. 206-209.) Gives results of test of long timber piles.

——Abstract. 1904. (*Minutes of Proceedings*, Inst. C. E., v. 158, p. 475-476.)

CARLIN, J. P. Progress of Work at the United States Naval Academy. 3 ill. 1901. (*Engineering Record*, v. 43, p. 449-452.) Gives results of tests on bearing value of piles made at Annapolis, Md.

COLBERG, OTTO. Eine Probelastung mit dem Betonpfahlgründungssystem "Strauss". 1909. (*Beton und Eisen*, v. 8, p. 54-58.) Describes "Strauss" system and tests in very soft, yielding soils.

CONCRETE PILE FOUNDATION OF THE U. S. EXPRESS CO. BUILDING, NEW YORK City. 1904. (*Engineering News*, v. 52, p. 348-349.) Gives tests for bearing value of Raymond system concrete piles.

COTTERILL, GEORGE F. Supporting Power of Piles. 1902. (*Engineering Record*, v. 45, p. 231-232.) Letter to editor giving results of tests on piles in Seattle.

FOLLANSBEE, ROBERT. Supporting Power of Piles Driven by a Steam Hammer After Standing. 1904. (*Engineering News*, v. 51, p. 542.) Short letter to editor giving test data.

GOW, CHARLES R. Concrete Piles. 1 dr., 3 ill. 1907. (*Journal*, Assoc. Eng. Soc., v. 39, p. 255-265.) Gives tests on bearing value of concrete piles.

——Abstract. 1907. (*Engineering News*, v. 59, p. 305-307.)

HOWE, HORACE J. Piles and Pile-Driving, New and Old. 1898. (*Journal*, Assoc. Eng. Soc., v. 20, p. 257-294.) Discussion, 1 ill., p. 294-312. Reviews a number of important papers on piles and pile driving. Gives results of many tests performed by various experimenters.

LOAD-TESTS ON CONCRETE PILES, NORTH SIDE POINT BRIDGE APPROACH, Pittsburgh. 1 diag., 1 ill. 1914. (*Engineering News*, v. 72, p. 310-311, 507, 510.)

PILE-DRIVING TEST. 1 ill. 1899. (*Engineering*, London, v. 68, p. 824, 826.) Loading tests at New Quay, Royal Victoria Dock, London.

RESULTS OF A 60-TON, TWO MONTHS LOAD TEST ON A CONCRETE PILE. 1911. (*Engineering-Contracting*, v. 36, p. 224.)

SANDEMAN, JOHN WATT. Experiments on the Resistance to Horizontal Stress of Timber Piling. 1 pl. 1879. (*Minutes of Proceedings*, Inst. C. E., v. 59, p. 282-285.)

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SCHUYLER, MONT. Kingshighway Viaduct, St. Louis, Mo. 1912. (*Engineering News*, v. 67, p. 1226-1233.) Importance of test loading for concrete piles, p. 1229.

TEST COMPARING STEAM AND DROP-HAMMER PILE FORMULAS. 1916. (*Engineering News*, v. 75, p. 33-34.) Nine piles were driven with drop hammer and eleven with steam hammer, in an area having soil of uniform consistency and therefore presumably of uniform bearing value. From data so obtained, comparisons were made of accepted bearing value formulas.

TEST OF BEARING POWER OF PILES. 1893. (*Engineering News*, v. 30, p. 3.) Tests were made before construction of Chicago Public Library Building. Considers nature and bearing value of soil. *See also* editorial, v. 31, p. 283.

TEST OF CONCRETE PILES. 1910. (*Engineering Record*, v. 62, p. 715.) Gives methods and results of tests.

TEST OF THE SAFE LOADS FOR PILES. 1902. (*Engineering Record*, v. 46, p. 84.) Note giving results of tests at New Quay, Royal Victoria Dock, London.

TESTS OF THE BEARING POWER OF PILES. 1894. (*Engineering News*, v. 31, p. 348.) Gives abstract of thesis of R. F. Gadd.

WELSH, J. J. Observations on Driving Piles with a Steam Hammer. 1904. (*Journal*, Assoc. Eng. Soc., v. 33, p. 193-197.) Compares results of test loadings of piles with formulas for bearing value.

——Condensed. 1904. (*Engineering News*, v. 52, p. 497.)

WELSH, J. J. Test Loads of Piles Driven with a Steam Hammer. 1904. (*Engineering News*, v. 52, p. 497.) Tests were made at a San Francisco site, in soft soil.

ZIMMERMANN, KARL. Rammwirkung im Erdreich, versuche auf neuer Grundlage. 1915. (*Beton und Eisen*, v. 14, p. 188-190, p. 205-209.) Interesting laboratory tests of the effect of piles on the surrounding earth. Small model piles, both blunt and pointed, were driven through strata separated by thin layers with distinctive coloring. After driving, the effect on displacement and compression of the adjacent earth was determined by a study of the vertical cross-section.

PILE-DRIVING.

BURNELL, G. R. Practical Observations on Pile Driving. 1855. (Papers Read at the Royal Inst. of British Architects, v. 5, p. 115-122.) Supporting power of wooden piles, p. 117.

CROWELL, J. FOSTER. Uniform Practice in Pile-Driving. 7 diag. 1892. (*Transactions*, Am. Soc. C. E., v. 27, p. 99-114, 129-172, 589-602.) Trautwine, Wellington, and others took part in the discussion. Gives a comprehensive review of the subject of bearing value of piles.

——Condensed. 1892. (*Engineering News*, v. 28, p. 398-400, 412-413, 438-440, 460-461.)

DE BURGH, ERNEST MACARTNEY. Pile-Sinking by Means of a Hydraulic Jet at Moruya and Carrington Bridges, New South Wales. 1 pl. 1902. (*Minutes of Proceedings*, Inst. C. E., v. 150, p. 340-351.) Tests of bearing power of piles, p. 348.

HAMMATT, W. C. Anomalous Resistance in Soft Mud; Effect of Hammer Shock. 1907. (*Engineering News*, v. 58, p. 173-174.) Letter to editor. Gives experience with pile-driving while building a wharf in San Francisco Bay.

HOWELL, C. S. Bottom Driven Concrete Piles on Government Job. 1916. (*Engineering News*, v. 76, p. 1207.) Piles driven by temporary follower against projections on pile point. Author also advocates designing concrete piles according to the load they will have to carry, rather than according to the usual haphazard methods.

JAMES, JOHN WILLIAM. On the Driving of Piles to Resist the Force of Ice Tending to Draw Them from the Ground. 1875. (*Minutes of Proceedings*, Inst. C. E., v. 41, p. 191-202.) Results of series of experiments.

LARGE CONCRETE PILE INSTALLATION; DATA ON PRELIMINARY BEARING Tests and Details of Driving more than 11 000 Cast-in-place Piles for Steel Plant Foundations. 1913. (*Engineering Record*, v. 67, p. 36-37.) Piles were driven through layers of granulated slag, loam, yellow clay, sand, sand and clay, and running sand, to rock.

RICHARDSON, P. A. Chief Features in Building a Long Concrete Viaduct in St. Louis. 1914. (*Engineering Record*, v. 70, p. 692-693.) Experiences in driving concrete piles with two types of hammers.

VAN AUKEN, A. M. Driving Piles. 1887. (*Railroad Gazette*, v. 19, p. 507.) Results of experiments in alluvial soil.

WHITTEMORE, D. J. On the Nasmyth Pile Driver. 1883. (*Transactions*, Am. Soc. C. E., v. 12, p. 441-443.) Shows that whenever the head of the pile becomes broomed, the effectiveness of any hammer is greatly lessened, due to elasticity of the pile head.

APPENDIX C

A STANDARD SCREEN SCALE FOR TESTING SIEVES.

Adopted by a Conference of Representatives of Various Scientific and Technical Societies, Government Bureaus, and Private Firms, held at the Bureau of Standards, and Recommended for General Adoption in the Interests of Securing Uniformity of Usage.

Since the adoption by the Bureau of Standards, several years ago, of specifications for standard 100- and 200-mesh sieves, frequent requests have been received that this Bureau test and certify sieves of other sizes. With a view to the adoption of a series of standard testing sieves which might be of use to all industries making fineness tests, this Bureau for two years has been studying the question of such a standard screen scale. Various scales that have been proposed were considered, and information was sought from representative firms, in the various industries interested, as to their requirements. Manufacturers of sieves have also been consulted as to the desirability of different screen scales and the practicability of their manufacture. As a result of this study of the question, a conference was called at the Bureau of Standards, on April 20th, 1916, of representatives of various committees of the American Society for Testing Materials, American Society of Civil Engineers, American Institute of Mining Engineers, American Foundrymen's Association, Mining and Metallurgical Society of America, American Water Works Association, American Institute of Metals, and the American Spice Trade Association; also representatives of the Committee of Revision of the U. S. Pharmacopoeia, the U. S. Geological Survey, the U. S. Bureau of Mines, the U. S. Office of Public Roads and Rural Engineering, the U. S. Office of Grain Standardization, and the U. S. Bureau of Standards; also representatives of a number of private firms engaged in industries in which sieves are used, such as the glass, the drug milling, the abrasive, the asphalt, the mining, the spice, the chemical, and the graphite industries; also representatives of the firms in this country manufacturing wire cloth and sieves.

This Conference, after considering the various screen scales either proposed or now in use, adopted as a Standard Screen Scale that given in Table 1, and recommended that it be adopted generally by scientific, technical, and engineering societies and committees, and by branches of National, State, and Municipal Governments as a part of their specifications for materials and methods of test; also that it be used by private firms who have need of standard sieves.

This screen scale is essentially metric. The sieve having an opening of 1 mm. is the basic one, and the sieves above and below this in the series are related to it by using in general the square root of 2, or 1.4142, or the fourth root of 2, or 1.1892, as the ratio of the width of one opening to the next smaller opening. The first ratio is used for openings between 1 mm. and 8 mm., the fourth root of 2 is used as the ratio for openings below 1 mm. to give more sieves in that part of the scale. The series has been made large enough, it is hoped, to meet the needs of all industries. Some industries may

TABLE 1.—STANDARD SCREEN SCALE.

Based on a 1-mm. opening sieve with the square root of 2, or 1.4142, as the ratio of the openings of successive sieves coarser than 1 mm., and the fourth root of 2, or 1.1892, as the ratio of the openings of successive sieves finer than 1 mm.

Size of sieve.	Opening.	Mesh.	Wire diameter.	Ratio of wire diameter to opening.	Tolerances.	
					Mesh.	Diameter.
8-mm.						
Metric	8.00	1	2.00	0.25	±0.01	±0.08
Customary	0.315	2.54	0.079	....	±0.03	±0.003
5.66 mm.						
Metric	5.66	1.4	1.48	0.26	±0.01	±0.08
Customary	0.223	3.56	0.058	....	±0.03	±0.003
4-mm.						
Metric	4.00	2	1.00	0.25	±0.02	±0.05
Customary	0.157	5.1	0.039	....	±0.05	±0.002
2.88-mm.						
Metric	2.83	2¾	0.81	0.29	±0.02	±0.05
Customary	0.111	7.0	0.032	....	±0.05	±0.002
2-mm.						
Metric	2.00	3.9	0.56	0.28	±0.04	±0.05
Customary	0.079	9.9	0.022	....	±0.1	±0.002
1.41-mm.						
Metric	1.41	5	0.59	0.42	±0.08	±0.025
Customary	0.0555	12.7	0.0232	....	±0.2	±0.0010
1-mm.						
Metric	1.00	7	0.43	0.43	±0.15	±0.020
Customary	0.0394	17.8	0.0169	....	±0.4	±0.0008
0.85-mm.						
Metric	0.85	8	0.40	0.47	±0.2	±0.015
Customary	0.0335	20.3	0.0157	....	±0.5	±0.0006
0.71-mm.						
Metric	0.71	9	0.40	0.56	±0.3	±0.012
Customary	0.0280	22.9	0.0157	....	±0.75	±0.0005
0.59-mm.						
Metric	0.59	10	0.41	0.69	±0.4	±0.012
Customary	0.0232	25.4	0.0161	....	±1.0	±0.0005
0.5-mm.						
Metric	0.50	12	0.33	0.66	±0.4	±0.012
Customary	0.0197	30.5	0.0130	....	±1.0	±0.0005
0.42-mm.						
Metric	0.42	14	0.29	0.69	±0.6	±0.010
Customary	0.0165	35.6	0.0114	....	±1.5	±0.0004
0.36-mm.						
Metric	0.36	16	0.26	0.72	±0.6	±0.010
Customary	0.0142	40.6	0.0102	....	±1.5	±0.0004
0.29-mm.						
Metric	0.29	20	0.21	0.72	±0.8	±0.010
Customary	0.0114	50.8	0.0083	....	±2	±0.0004
0.25-mm.						
Metric	0.25	23	0.185	0.74	±1	±0.008
Customary	0.0098	58.4	0.0073	....	±3	±0.0003
0.21-mm.						
Metric	0.21	27	0.16	0.76	±1	±0.008
Customary	0.0083	68.6	0.0063	....	±3	±0.0003
0.17-mm.						
Metric	0.17	31	0.15	0.88	±1	±0.008
Customary	0.0067	78.7	0.0059	....	±3	±0.0003
0.14-mm.						
Metric	0.14	39	0.116	0.83	±1	±0.008
Customary	0.0055	99.1	0.0046	....	±3	±0.0003
0.125-mm.						
Metric	0.125	47	0.089	0.71	±1.5	±0.008
Customary	0.0049	119.4	0.0035	....	±4	±0.0003

TABLE 1.—(*Continued.*)

Size of sieve.	Opening.	Mesh.	Wire diameter.	Ratio of wire diameter to opening.	Tolerances.	
					Mesh.	Diameter.
0.105-mm.						
Metric	0.105	59	0.064	0.61	±2	±0.008
Customary	0.0041	149.9	0.0025	....	±5	±0.0003
0.088-mm.						
Metric	0.088	67	0.061	0.69	±2.5	±0.005
Customary	0.0035	170.2	0.0024	....	±6	±0.0002
0.074-mm.						
Metric	0.074	79	0.053	0.72	±3	±0.005
Customary	0.0029	200.7	0.0021	....	±8	±0.0002
0.062-mm.						
Metric	0.062	98	0.040	0.65	±3.5	±0.005
Customary	0.0024	248.9	0.0016	....	±9	±0.0002
0.052-mm.						
Metric	0.052	110	0.039	0.72	±4	±0.004
Customary	0.0021	279.4	0.0015	....	±10	±0.00015
0.044-mm.						
Metric	0.044	127	0.035	0.80	±5	±0.004
Customary	0.0017	323	0.0014	....	±12	±0.00015

have occasion to use all the sieves in a certain section of the series and none of the others; in other industries it may be desirable to use only certain sieves selected from the whole range of the series. In making such selections, it is recommended that this be done on some systematic plan as, for example, the selection of every other sieve or of every fourth one in the series below 1 mm. opening, and every other sieve above 2 mm., in which case the ratio of each opening to the next smaller one would be as 2 to 1.

Because of the wide range of openings in sieves now manufactured which are possible with a given number of meshes of wire per unit of length by the use of wires of different diameters, and the consequent confusion and uncertainty which arises in designating sieves by the number of meshes per unit of length, the sieves of this series have been designated by the width of the opening, in millimeters, as, for example, a 1.41-mm. sieve, or a 0.36-mm. sieve. It is urgently recommended that all users of sieves in the future designate these standard sieves in this way, and that the manufacturers mark and list the sieves in this manner rather than by the meshes per inch.

In the designation and certification of the sieves, the metric units will be used by the Bureau of Standards. In Table 1, however, are also given the equivalents of these metric quantities, in inches, in order that the series may be more readily related to work previously done. It is immaterial, of course, whether units of the metric system, or of the customary system, or of any other system, are used in the manufacture of the sieves, provided they are within the tolerances.

To meet the need for sieves of the series at the present time, a temporary provision has been made in the specifications for the acceptance of sieves of slightly different mesh and wire diameter than that called for in the screen scale, provided the resultant opening is the same as the nominal opening, within a small range. This will make

possible the use of a number of sieves now on the market in which the ratio of wire diameter to opening is only slightly different from that of the screen scale. This provision will be withdrawn when conditions are such that manufacturers can furnish sieves made more exactly in accordance with the specifications.

The Bureau of Standards hereby announces that it will test sieves of this series to determine whether they conform to the specifications. This test will consist of the examination of the mesh, of both the warp and shoot wires of the cloth, to ascertain whether it comes within the tolerances allowed; also measurements of the diameter of wires in each direction, to determine the average diameter, and a measurement of any large openings to ascertain whether they exceed the limits given in these specifications; also an examination of the sieve to discover any imperfections which may affect seriously its sieving value. Sieves which pass the specifications will be stamped with the seal of this Bureau, and will be given an identification number, and a certificate will be furnished for each sieve that passes the requirements.

For sieves which fail to meet the specifications, reports will be rendered showing wherein the sieve was not up to the standard.

A fee of $2.00 per sieve will be charged for the test of the sieves when submitted singly. For from 2 to 9 sieves submitted at one time the fee will be $1.50 per sieve. For lots of 10 or more the fee will be $1.00 per sieve. Only half of the above fees will be charged for such sieves as may be rejected for exceeding the tolerances of mesh, in which case the wire diameter will not be measured.

In Table 1 there is given in the first column, headed "Openings", the width of the opening (on the first line in millimeters, on the second line in inches), for each sieve. In the second column, headed "Mesh", on the first line is given the number of meshes per linear centimeter, and on the second line the equivalent number of meshes per linear inch. In the third column, headed "Wire diameter", is given on the first line the diameter of the wire in millimeters, and on the second line its equivalent in inches. In the fourth column, headed "Ratio of wire diameter to opening", is given the ratio of the wire diameter to the width of the opening between the wires. In the fifth and sixth columns, headed "Tolerances", are given the tolerances for these sieves mentioned in the specifications. These tolerances—for testing purposes—will be used essentially in the metric dimension, but on the second line in each case is given their equivalent in inches, in order that they may be compared readily with tolerances in previous use. The tolerance in the fifth column is that for the meshes per centimeter and per inch, respectively, and in the sixth column the tolerances for wire diameter, in millimeters and inches, on the first and second lines, respectively.

In Table 2 is given a list showing the dimensions of sieves now on the market which would most nearly meet the specifications and tolerances of the Standard Screen Scale. The headings of the columns of this table are self-explanatory. Where the dimensions of more than one sieve are shown for a given sieve of the screen scale, one set of dimensions are those of one manufacturer, and the other those of another. In some cases the third set, if one is given, are made by two or more manufacturers.

TABLE 2.—Sieves Now on the Market Which Would Most Nearly Meet the Tolerances of the Standard Screen Scale.

Size of sieve.	Opening, in millimeters.	Opening, in inches.	Meshes per inch.	Wire diameter, in inches.	Wire diameter, in millimeters.
8-mm	8.13	0.320	2.5	0.080	2.03
	8.05	0.317	2.5	0.083	2.11
5.66-mm	5.66	0.223	3.5	0.063	1.60
	5.61	0.221	3.5	0.065	1.65
4-mm	4.04	0.159	5	0.041	1.04
	4.06	0.160	5	0.040	1.02
2.83 mm	2.82	0.111	7	0.032	0.81
	2.82	0.111	7	0.0315	0.80
2-mm	1.96	0.077	10	0.023	0.58
	2.03	0.080	10	0.020	0.51
	2.03	0.080	10	0.0205	0.52
1.41-mm	1.40	0.055	12	0.028	0.71
	1.42	0.056	12	0.027	0.69
1-mm	1.01	0.0396	18	0.016	0.41
	0.99	0.0391	18	0.0165	0.42
0.85-mm	0.85	0.0335	20	0.0165	0.42
	0.86	0.0340	20	0.016	0.41
0.71-mm	0.72	0.0285	22	0.017	0.43
	0.70	0.0275	22	0.018	0.46
	0.74	0.0290	22	0.0165	0.42
0.59-mm	0.58	0.0230	26	0.0155	0.39
	0.60	0.0235	26	0.015	0.38
0.5-mm	0.50	0.0198	30	0.0135	0.34
	0.50	0.0196	30	0.01375	0.35
0.42-mm	0.42	0.0166	35	0.012	0.30
	0.42	0.0164	35	0.01225	0.31
0.36-mm	0.36	0.0140	40	0.011	0.28
	0.35	0.01375	40	0.01125	0.29
	0.37	0.01475	40	0.01025	0.26
0.29-mm	0.28	0.0110	50	0.009	0.23
0.25-mm	0.25	0.0097	60	0.007	0.18
	0.26	0.0102	60	0.0065	0.17
	0.23	0.0092	60	0.0075	0.19
0.21-mm	0.19	0.0073	70	0.007	0.18
	0.20	0.0078	70	0.0065	0.17
0.17-mm	0.17	0.0068	80	0.00575	0.15
0.14-mm	0.14	0.0055	100	0.0045	0.114
0.125-mm	0.117	0.0046	120	0.0037	0.094
	0.119	0.0047	120	0.0036	0.091
0.105-mm	0.104	0.0041	150	0.0026	0.066
	0.094	0.0037	150	0.0030	0.076
0.088-mm	0.089	0.0035	170	0.0024	0.061
	0.084	0.0033	170	0.0026	0.066
0.074-mm	0.074	0.0029	200	0.0021	0.053
0.062-mm	0.061	0.0024	250	0.0016	0.041
	0.058	0.0023	250	0.0017	0.043
0.052-mm	0.051	0.0020	280	0.0016	0.041
0.044-mm	0.041	0.0016	325	0.0015	0.038
	0.041	0.0016	330	0.0014	0.036

In Table 3 is given a list of sieves between 1 and 8-mm. openings, which would be interpolated in the series of the Standard Screen Scale if the fourth root of 2, or 1.1892, were used as the ratio of successive sieves throughout the series. Suitable meshes and wire diameters to give these openings are also given, together with the tolerances under which such sieves would be tested if used. These sieves have not been included in the Standard Screen Scale, as it is believed to be unnecessary to have so many sieves in this part of the scale. This list is given separately, however, in case any organization, in selecting sieves systematically from the Standard Screen Scale, in the series of openings less than 1 mm., finds it desirable to use any of these interpolated sieves about 1 mm. in carrying out their systematic plan of selection of sieves. In case any organization or firm should adopt any of these six sieves, under such circumstances, the Bureau of Standards will test and certify them in accordance with the dimensions given herewith.

TABLE 3.—Additional Sieves Which Would be Interpolated Between the 8-mm. Opening and the 1-mm. Opening of the Standard Screen Scale by the Use of the Fourth Root of 2, or 1.1892, as the Ratio of the Successive Sieve Openings.

Size of sieve.	Opening.	Mesh.	Wire diameter.	Ratio of wire diameter to opening.	Tolerances:	
					Mesh.	Diameters.
6.72-mm.						
Metric	6.72	1.2	1.61	0.24	± 0.01	± 0.08
Customary	0.265	3.05	0.063	....	± 0.03	± 0.003
4.76-mm.						
Metric	4.76	1.6	1.49	0.31	± 0.02	± 0.05
Customary	0.187	4.1	0.059	....	± 0.05	± 0.002
3.36-mm.						
Metric	3.36	2.4	0.81	0.24	± 0.02	± 0.05
Customary	0.132	6.1	0.032	....	± 0.05	± 0.002
2.38-mm.						
Metric	2.38	3.15	0.79	0.33	± 0.04	± 0.05
Customary	0.094	8.0	0.031	....	± 0.1	± 0.002
1.68-mm.						
Metric	1.68	4	0.82	0.49	± 0.04	± 0.025
Customary	0.067	10.2	0.032	....	± 0.1	± 0.0010
1.19-mm.						
Metric	1.19	6	0.48	0.40	± 0.1	± 0.020
Customary	0.0468	15.2	0.0189	....	± 0.25	± 0.0008

In Table 4 is given an extension of the metric series beyond the 3-mm. opening, using the square root of 2, or 1.4142, as the ratio of successive openings. A suggested diameter of wire for use in making such sieves is also given. No tolerances for these sieves are given, however, as it is not proposed to test such sieves at the Bureau of Standards, since the user of the sieves could ordinarily make such tests as are necessary with sufficient accuracy with such means as he may have at hand. Such sieves would generally be made up into sieves of larger diameter than those of the Standard Screen Scale, and would usually be made of iron or steel wire.

TABLE 4.—Showing the Sieves which an Extension of the Standard Screen Scale from an 8-mm. Opening to a 128-mm. Opening would Comprise, Using the Square Root of 2, or 1.4142, as the Ratio of Successive Openings.

Size of sieve.	Opening.	Wire diameter.	Size of sieve.	Opening.	Wire diameter.
128-mm.			32-mm.		
Metric............	128	9.5	Metric...........	32.0	4.85
Customary.......	5.04	0.375	Customary......	1.26	0.192
90.5-mm.			22.6-mm.		
Metric............	90.5	9.5	Metric	22.6	4.11
Customary.......	3.56	0.375	Customary......	0.891	0.162
64-mm.			16-mm.		
Metric............	64.0	6.4	Metric..........	16.0	3.05
Customary.......	2.52	0.25	Customary......	0.630	0.120
45.3-mm.			11.3-mm.		
Metric............	45.3	5.26	Metric...........	11.3	2.67
Customary.......	1.78	0.207	Customary......	0.445	0.105

In the specifications the diameter or other dimensions of the sieve frames are not given, with the idea that any organization or firm, in adopting these specifications, will decide the size of sieve frame that best meets its needs. For purposes of uniformity and interchangeability of sieves, pans, and covers, it is recommended that sieves be purchased in diameters of either 20 cm., 15 cm., or 10 cm., (7.87 in., 5.91 in., or 3.94 in.). These are the outside diameters of the bottom of the sieve or the inside diameter of the top of the sieve.

Specifications for Sieves of the Standard Screen Scale.

Wire cloth for standard sieves shall be woven (not twilled, except that the cloth of 0.062-mm., the 0.052-mm., and the 0.044-mm. sieve, may be twilled until further notice) from brass, bronze, or other suitable wire, and mounted on the frames without distortion. To prevent the material being sieved from catching in the joint between the cloth and the frame, the joint shall be smoothly filled with solder, or made so that the material will not catch.

The number of wires per centimeter of the cloth of any given sieve of the Standard Screen Scale shall be that shown in Table 1, in the second column, headed "Mesh", and the number of wires in any whole centimeter shall not differ from this amount by more than the tolerance given in the fifth column, that headed "Mesh" under the heading "Tolerances." No opening between adjacent parallel wires shall be greater than the nominal width of opening for that sieve by more than the following amounts:

10% of the nominal width of opening for the 8-mm. to 1-mm. sieves, inclusive.

25% of the nominal width of opening for the 0.85-mm. to the 0.29-mm. sieves, inclusive.

40% of the nominal width of opening for the 0.25-mm. to the 0.125-mm. sieves, inclusive.

60% of the nominal width of opening for the 0.105-mm. to the 0.044-mm. sieves, inclusive.

The diameters of the wires of the cloth of any given sieve shall be that shown in the third column of Table 1, headed "Wire diameter"; and the average diameter of the wires in either direction shall not differ from the specified diameter by more than the tolerance given in the last column of Table 1, that under "Tolerances" headed "Diameter."

The Bureau of Standards also reserves the right to reject sieves for obvious imperfections in the sieve cloth or its mounting, as, for example, punctured, loose, or wavy cloth, imperfections in soldering, etc.

Until further notice, to permit the use of sieves now on the market which have slightly different mesh and wire diameters from those specified herein, sieves will be certified as satisfactory if the measurements of mesh and wire diameters show the resulting average width of opening to be within 4% of the nominal opening of a given sieve, and the ratio of wire diameter to opening of the sieve in question is within 0.03 of that given in Table 1 in the column headed "Ratio of Wire Diameter to Opening" for the 8-mm. to the 2-mm. sieves, inclusive, and within 0.06 of the ratio given for sieves of smaller openings than 2 mm.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

A METHOD OF DETERMINING A REASONABLE SERVICE RATE FOR MUNICIPALLY OWNED PUBLIC UTILITIES

Discussion.*

By J. B. Lippincott, M. Am. Soc. C. E.†

J. B. Lippincott,‡ M. Am. Soc. C. E. (by letter).§—Almost without exception, the discussion brought forth by this paper agrees with the writer in that the service charges for a publicly owned public utility should be fixed on the same basis as those of a privately owned utility which is subject to rate regulation. Mr. Lippincott.

Mr. Harris has indicated the basic element when he says:

"It is only as a result of such an equitable attitude that private capital may be expected to 'pioneer' in those fields into which the public is not for the time desirous of entering."

It is not only that the present investment of private capital in public utilities should be protected, but that the policies adopted be such that the further investment in utility enterprises in undeveloped communities be fostered and encouraged by the assurance that these investments have the sanction and protection of legislative bodies. Mr. Yereance has also called attention to this point. Mr. Yereance and Mr. Thomson have discussed to some extent the relative merits and efficiencies of municipally operated utilities as against those privately operated, from which conclusions are drawn that it is difficult to obtain in municipally operated plants competent executives. It is generally not possible to pay salaries adequate to obtain the services of such men, and the executives are so hampered and tied with the red tape of

* Discussion of the paper by J. B. Lippincott, M. Am. Soc. C. E., continued from January, 1917, *Proceedings*.

† Author's closure.

‡ Los Angeles, Cal.

§ Received by the Secretary, June 4th, 1917.

Mr. Lippincott.

municipal and political organizations that effective results are not generally obtained. Because of unreasonable abuse it is not a sufficient honor to hold a municipal office. Although this may be the general rule, yet some notable exceptions have come under the writer's observation. The Water Department of Los Angeles is most efficiently organized and operated. Santa Barbara and Long Beach, Cal., both own and operate their water-works. The management is efficient, and the earnings of the departments show a substantial profit.

Mr. Jordan's discussion is more fanciful than consistent. The relative merits of private or municipal ownership of public utilities is not at issue. The effort has been made in the paper to apportion the expense of a publicly owned utility so that each member of the community will carry his proportion and that private enterprise may not be discouraged. The segregation of the expenses is largely a matter of judgment.

Mr. Clark suggests that the consumers should pay through their rates only for the maintenance and operation of the system and the interest on the bonds, the depreciation being derived from the general tax levy. Relative to the redemption of bonds, Mr. Clark also states:

"If the city makes a 7% interest charge, as would be granted to a privately owned utility, any profit resulting under the bond rate of interest could be laid aside to redeem the bonds. If the bond rate of interest is 5%, the differential of 2% compounded annually at 4% will approximately retire the bonds in 30 years."

In the writer's classification of expense accounts, the effort has been made to create a service rate which would be in harmony with that for a privately owned utility. Under Mr. Clark's segregation of expenses, the service rates would be below those which could be profitably charged by a private corporation. Rates for privately owned utilities are fixed with a view of yielding the owner a fair profit on his investment. If this is done with a municipally owned plant the "fair profit" could be turned over to the general tax fund of the city. If the bond redemption fund is paid from the general tax levy and the profits accruing from the operation of the system are paid into the general treasury of the city, it appears to the writer that the same end is achieved whether the bond redemption fund is carried by the utility department itself or by the city at large.

There is a surprising tendency constantly to put more burden on the property owner and less on the thriftless individual, whether that ownership be of realty or of a utility. The result tends to discourage investment in enterprise and to encourage indolence. Mr. Irving and Mr. Clark both consider that the owner of vacant property should be exempt from any special tax due to the utility other than that imposed on the city at large. It is the writer's contention that the benefits

Mr. Lippincott.

derived by the owner of vacant lots, because of the fact that the service of the utility is immediately available, is out of proportion to the charges on the property through taxes, and that, in addition to these charges, the owner should pay a special tax, probably on a front-foot basis. These fees should be considered as an addition to surplus, and could be turned into the general fund of the city. They have no counterpart in a privately owned utility, and would operate to the benefit of the community at large. Mr. Clark states: "It should not be the aim of a city to make its water consumers pay more than the cost of the service." It appears to the writer that a municipally owned utility should be viewed as a community investment and should be operated at a profit in the same way as a corporation is operated for the profit of its stockholders. The property owners are in effect stockholders in the municipal plant. Mr. Clark's contention concerning the depreciation of pipe lines in front of vacant property is true. Rate regulating bodies in California have adopted a practice of reducing the capital and depreciation accounts of private utilities where they appear to be overbuilt. Some such system might be adopted here to relieve the consumers from the depreciation charges for mains paralleling vacant property, and these charges might be made up from the proposed special tax on vacant lots.

Mr. Whitney's segregation of expenses is extremely interesting, and appears to the writer to be entirely logical. However, in the case of the competing private enterprise, it is doubtful if municipal legislative authorities would consent, possibly for political reasons, to the inclusion in the annual tax budget of an assessment to cover interest on the value of a privately owned plant.

Although it is not proposed to review the procedure of rate regulating commissions for private utilities, the writer might add, to what Mr. Hazen has said, that these commissions would increase their economic value to the community if they evolved a plan under which a premium was placed on efficient management of the utilities coming under their jurisdiction.

In closing, the writer wishes to express his appreciation of the discussion called out by the paper. He feels that the public welfare would be better served if policies were adopted by those in authority in legislative positions which would encourage and foster the further investment of private capital, particularly in those States which are in a formative or developing process. It should be kept well in mind that legislation or Court decree cannot compel the independent investor who is seeking new enterprises to enter fields where others already operating on such lines are losing money due to unfair competition.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

THE VALUATION OF LAND

Discussion.*

By Messrs. Franklin F. Mayo and L. P. Jerrard.†

Franklin F. Mayo,‡ Esq.—The value of land is not constant, but is always moving up or down in the scale, with a period of stationary value, the time element of these three phases being uncertain, owing to changes in the factors producing the value. Also, certain locations and districts possess a premium value over the income value, at certain periods. The speaker contends that this premium value is uncertain, and should be considered in all taxing valuations, as a property in a district where values are declining should be taxed (that is, valued) at a price based on the income obtainable on a lease made at that time, and not on a basis of sale during the premium period. Mr. Mayo.

To do this would require a keen sense of the human element factor, and no method of valuation would be satisfactory unless made by men especially qualified in placing values.

L. P. Jerrard,§ Jun. Am. Soc. C. E. (by letter).‖—The discussion of this paper has brought out several points which may be cleared up by further explanation. The writer's loose application of economic terms has evidently obscured his argument in several instances. Mr. King and Mr. Kelly both take exception to the statement: "The true value of land is the ground rent capitalized." Inasmuch as land has Mr. Jerrard.

* Discussion of the paper by L. P. Jerrard, Jun. Am. Soc. C. E., continued from March, 1917, *Proceedings*.

† Author's closure.

‡ Newark, N. J.

§ Chicago, Ill.

‖ Received by the Secretary, May 4th, 1917.

Mr. Jerrard. value because it is a source of ground rent, it would seem that the market value should bear some definite relation to the amount of ground rent. Such is not the case, in view of the fact that the ground rent is subject to fluctuation, and market values discount the prospective changes in the amount of ground rent. The intention was to bring out the distinction between market value and a value based entirely on ground rent capitalized, which was inaccurately designated by the term "true value". The statement, "The basis of business values is strictly economic, * * * but the basis of residence values has a social element," is also challenged by Mr. King, who states that the demand of wealthy people for pleasant residence sites is as strictly economic as the demands of any business utility for land. The social element referred to by the writer manifests itself in the demand of people of lesser wealth to live as near as possible to the wealthiest class. Of two residence sites, all other factors (such as schools and transportation facilities) being equal, the one nearest the best residence section of the community is ordinarily preferred. It would seem as though this tendency is more sentimental than economic, but, in either case, it was the distinction between business and residence values which was intended to be made.

Mr. Kelly has brought up several points to which the writer must take exception. To the deductions to be made from gross rentals in computing ground rent Mr. Kelly would add, "A sinking fund charge sufficient, when put out at interest, to cover the cost of the building in a given number of years." This charge is covered by the item "annual depreciation" which was included in the list of deductions. "Advantages of location govern the values entirely." By "values", as used in this statement, is meant values in general, or unit values, which would eliminate the factors of shape and size suggested by Mr. Kelly. He also suggests as a factor governing values, "utility", but does not utility depend entirely on location? Land has greater value when utilized for business purposes than for residence property; but it cannot be used for business purposes unless it is in the business section of a community, and even then its value depends on its particular location within the business section.

Mr. Kelly understands the writer to say that the greatest number of people pass through the heart of the city. Possibly he has in mind the statement, "retail stores gather at the heart of the city" where the largest number of prospective customers will pass. It is frequently true, as Mr. Kelly states, that the greatest number of people pass some railroad terminal or ferry landing, but they are hurrying to and from their work and are not "prospective customers." It may be interesting to note that such corporations as the United Cigar Stores Company, and F. W. Woolworth Company, before locating one of their stores,

make a careful investigation, not only of the number of people passing a prospective location, but of their wants, buying power, and inclination to buy at that point. It is possible that a cigar store would flourish at a terminal where no one would find time to patronize a five and ten-cent store. A traffic count might show a great number of people passing a certain corner, but, if they were mostly factory girls and women, it would not augur well for a cigar business. Mr. Jerrard.

"The retail section moves in the direction of the best residence section." It is true, as Mr. Kelly suggests, that local conditions sometimes alter or retard this tendency, but the tendency persists nevertheless. The writer noted an interesting situation recently in Watertown, S. Dak. The sketch, Fig. 8, is from memory only and not accurate, but illustrates the situation approximately. The business section is

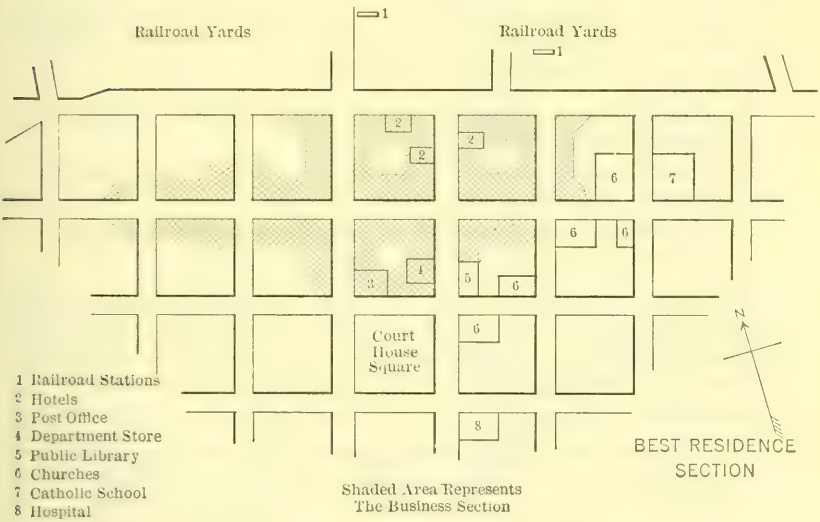

FIG. 8.

separated from the best residence section by a zone of public buildings, churches, and other permanent institutions, which must form a substantial barrier to the extension of the business section in that direction. To the west and north, where there is room for the expansion of the business section, there is nothing more attractive to business than railroad yards and river bottoms. The tendency of retail business to reach out toward the best residence section seems to be illustrated by the location of a large department store, the best in the city, on the edge of the business section, but on this side most convenient to the best residence district.

Mr. Jerrard. Mr. Walker speaks of the Somers system of assessing cities, with the aid of community opinion. In fixing the point of greatest value at 100%, and determining the value of all surrounding frontage in percentages thereof, as explained by him, it would seem that all errors which may be introduced are cumulative. If an incorrect relative value is established for any block, the error is reflected in a series of blocks the values of which are relative to the one in error. The writer has seen evidence of the partial failure of the Somers system in this respect, and has also noted that, in the public meetings and discussions, the principal efforts are made by those land owners who are most directly interested, and that the conclusions are likely to be biased in their favor.

Mr. Rankin has introduced some interesting and pertinent data with regard to the value of the best business land in cities. With respect to the discrepancy between the points plotted by Mr. Rankin and the curves advanced by the writer (Fig. 2), the following influences are suggested as regards Mr. Rankin's data:

1.—Population of cities does not include tributary suburbs (mentioned by Mr. Rankin);

2.—Undue optimism which frequently prevails among real estate dealers regarding land values;

3.—Difficulty of determining, in the business districts of a city, values which are free from corner influences.

All these factors would tend to throw the points plotted by Mr. Rankin away from the curves. Part of the data on which these curves were based, are here submitted, and also shown in Fig. 9. Mr. Alfred D. Bernard, in "Some Principles and Problems of Real Estate Valuation", states as follows:

"In a normal city for populations above 50 000 the value of the best retail business property is one cent per front foot per person.

"City.	Population.	Best Land per Front Foot.
Boston	1 500 000	$15 000.00
Philadelphia	1 500 000	15 000.00
Baltimore	650 000	6 000.00

"Baltimore has a large negro and low wage earning population and values are below normal. Cleveland, Louisville and Minneapolis will prove the rule."

Mr. Richard M. Hurd* has given the figures reproduced in Table 9.

From miscellaneous sources, including some of the writer's own determinations, city assessments by the Somers method, and opinions of real estate dealers, the data in Table 10 have been collected.

* In "Principles of City Land Values."

TABLE 9.

Mr. Jerrard.

City.	Population.	Best business per front foot.
Washington	270 000	$5 000
Louisville	205 000	1 700
Minneapolis	203 000	2 500
Indianapolis	169 000	2 500
Kansas City	164 000	2 500
St. Paul	163 000	1 800
Denver	134 000	1 800
Toledo	132 000	2 000
Memphis	102 000	2 000
Portland, Ore	90 000	1 600
Atlanta	90 000	2 000
Richmond	85 000	1 800
Seattle	81 000	2 000
Des Moines	62 000	1 500
Salt Lake City	54 000	1 400
Duluth	53 000	1 000
Spokane	37 000	800
	25 000	$300 to $400
	50 000	600 to 1 000
	100 000	1 200 to 2 000
	150 000	1 500 to 2 500
	200 000	1 800 to 3 000
	300 000	2 500 to 4 500
	600 000	4 000 to 7 000
	1 000 000	7 000 to 10 000

TABLE 10.

City.	Population.	Value of best business per front foot.
Boston, Mass	1 000 000	$15 000
Cleveland, Ohio	580 000	5 500
Milwaukee, Wis	420 000	6 000
Indianapolis, Ind	250 000	3 000
Des Moines, Iowa	94 000	2 000
St. Paul, Minn	230 000	2 500
Minneapolis, Minn	325 000	3 500
Superior, Wis	45 000	700
Racine, Wis	40 000	600
Madison, Wis	30 000	2 000

In Mr. Thomson's opinion, this paper suggests in too strong a manner that the engineer is a better authority on real estate than the real estate man. There is no implication that the engineer has any place in the general field of the real estate business, which includes platting, buying, selling, renting, and managing property, and the question pertains only to making land valuations. In valuation, it is felt that some of the work of real estate men is inaccurate, and that, in many cases, some effort should be made to check the appraisals submitted by them. This is especially the case in smaller cities. As

Mr. Jerrard. an indication that many valuations made by real estate men are in error, the writer may cite the frequent instances of wide variations in the valuation of the same property by different appraisers. The writer has had some experience, in cities of less than 50 000 population, in obtaining two valuations of the same property from the same man at intervals of several weeks or months. It is not uncommon to find considerable variation in the results in such cases. If a deliberate attempt is made to confuse the real estate man by argument, or by introducing units of value with which he is unfamiliar, a very wide variation in results may be obtained under the same circumstances. In all these cases, an effort was made to obtain the services of the best informed man in the community.

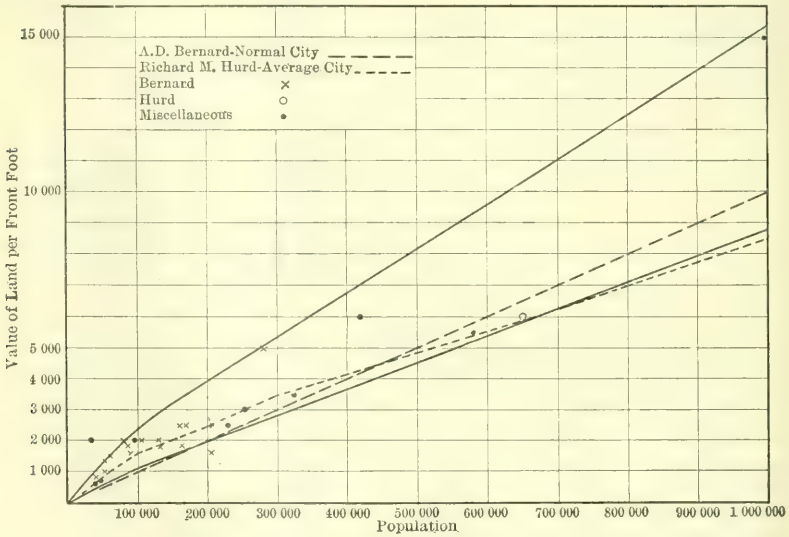

FIG. 9.

Some organizations, which require land valuations to be made in different cities, proceed in the following manner: A complete record of real estate sales is obtained from abstractors and from public records. The services of a local real estate man are obtained, and he states what unit of value is most familiar to him, such as the lot, square foot, or front foot. The sales data are then reduced to the same unit and plotted on a map, together with information collected as to the extent of improvements on properties sold, assessment records, etc. The map is then turned over to the expert, who studies the situation and makes the valuation of the land in question. Satisfactory results are obtained in this manner. The experience of such organizations has been that, if the data are not provided for the expert, he will not obtain

all of them, nor will he reduce them to the form in which they are most applicable to the problem. If the information is expressed in units with which he is unfamiliar, he cannot readily understand it or convert it to another unit. It is not unlikely, in cases such as these, that the man who collects and prepares the data becomes as familiar with the values involved as the local expert, but, in a contest, the local expert is the only one who can qualify as a witness. Mr. Jerrard.

A true real estate expert is a man who has had experience in land values, good judgment of actual conditions, foresight into tendencies, freedom from prejudice, and natural honesty. The field is one for a specialist, but it has not been developed outside of the largest cities. The average real estate dealer is not a trained man. He does not systematically collect and classify data with regard to values. He is not familiar with the different units of value, or with the graphical representation of data on the maps. He is hampered most of all by the fact that he is a trader in real estate, and must be a "booster", in order to make his living. Aside from books by the representatives of several large mortgage and bond companies, and the work of certain men engaged in taxation problems, the literature on the subject of land valuation is meager. In the writings of real estate dealers, terms such as "plottage", are rarely defined, and are used in a contradictory manner by different authors. Under the circumstances, the writer feels that the engineer in charge of a valuation may sometimes find occasion to investigate with his own staff the item of land, without encroaching on a field adequately covered by men of another profession.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

CONSTRUCTION METHODS FOR ROGERS PASS TUNNEL

Discussion.*

By Messrs. J. G. Sullivan and A. C. Dennis.†

J. G. Sullivan,‡ M. Am. Soc. C. E. (by letter).§—The object of this discussion is to correct some erroneous accounts and misstatements which have developed in previous discussions. The most flagrant of the latter was the statement, by Mr. S. A. Knowles,‖ that the credit for the quick work done at Rogers Pass Tunnel was due to Mr. McIlwee, who was the original contractor and was responsible for the methods and organization. This is very far from the truth. The facts of the case are as follows: Mr. Sullivan.

During January, February, and March, 1913, while the engineers were working on plans for the building of this tunnel, the writer had two or three conferences with Mr. Dennis, whose office at that time was in Winnipeg, and, on March 13th, 1913, he reported his ideas on the subject to the management of the Company in the following terms:

"Referring to the progress that we hope to make in driving of Rogers Pass Tunnel. I advised you in my report of October 22d regarding the relative speeds of driving tunnels on the American continent compared with those that have been driven through the Alps. I have given the matter considerable study since, and have come to the conclusion that the European method of driving a small lower heading and stoping out the remainder of the tunnel would be too expensive on this side, on account of the difference in the cost of labor.

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* Discussion of the paper by A. C. Dennis, M. Am. Soc. C. E., continued from May, 1917, *Proceedings*.

† Author's closure.

‡ Winnipeg, Man., Canada.

§ Received by the Secretary, May 14th, 1917.

‖ *Proceedings*, Am. Soc. C. E., April, 1917, p. 615.

Mr. Sullivan. "I have been thinking out and studying methods that would tend to expedite this work. I first thought of driving a heading in the center of the tunnel, about 9 ft. by 12 ft., and keeping this heading close to the bench, carrying the air pipes over the muck in front of the steam shovel and into the heading. I still believe that this method in rock that will stand, is better than an upper heading. Mr. A. C. Dennis, however, suggested driving a pioneer tunnel and taking out an upper heading through shafts into this tunnel, taking out the rest of the bench with steam shovels. I pointed out to him that this was impracticable for the reason that from an upper heading you cannot drill to the bottom of the tunnel, and therefore would have to clean up all the muck in the bench before you could put in a round of breast holes to break more rock. I have now made plans showing a combination of my ideas and Mr. Dennis', which I think is well worth studying. The plan is to drive a small working pioneer tunnel, 8 ft. by 8 ft. underneath the main tunnel. I am sending you this for your information and, further, if this method should be adopted, that Mr. A. C. Dennis may have the proper credit for first suggesting a pioneer tunnel."

This was about a year before the writer ever saw or ever heard of Mr. McIlwee. So much for the methods adopted in the construction of this tunnel.

As to the statement that Mr. McIlwee was the original contractor: Mr. McIlwee never had any contract with the Canadian Pacific Railway Company in connection with this work. In the fall of 1913, the general contractors sub-let a portion of the work of driving the pioneer headings and the center heading to Mr. McIlwee. There immediately appeared, in some Denver papers, a notice to the effect that Mr. McIlwee had a contract for a 5-mile tunnel in Canada. The following is a quotation from the writer's report to the Management as to what was meant by these notices:

"The understanding was that he would bring men from Colorado and drive the outside pioneer tunnel at so much per foot at a certain rate of speed. The contractors were to pay him a bonus on any excess over this specified rate of speed. Contractors to furnish the cars, air, steel, and all other plant, as I wired you before, the result being that these men were working at piece work. If they do not make the speed guaranteed, they can be put off the work and nobody will suffer, as the entire plant is owned and controlled by the contractor. A man without one cent but brains, can enter into an agreement of this kind, while it takes capital to enter into a contract to build a five-mile tunnel."

In other words, Mr. McIlwee, on this particular work, might be ranked with the grade of contractors known as stationmen on railway construction work.

In reference to the discussion by Mr. E. Lauchli: Mr. Lauchli, in attacking this method of doing work, quotes the writer as authority for prices bid, without giving the full quotation, and leaving an incorrect

impression. In the article, from which the writer thinks the quotation was taken,* the full paragraph reads as follows: Mr. Sullivan.

"I may add further that, in reply to my invitation of April 8, 1913, the contractors who are doing the work, having in mind the method which was later adopted and which was suggested by myself and one of their Engineers, bid $6.10 per cu. yd., with a time limit of 42 months from date of signing contract. Other responsible and supposedly expert tunnel contractors bid from $8 to $11.25 per cu. yd., with time limits varying from 42 to 48 months. I do not know what method these latter contractors proposed to employ, but I always presumed it would be some modification of the European method."

All contractors had the same information, and the majority of those who bid sent men on the ground where they could see the character of the rock, from unbroken rock exposure, for practically the entire distance of the tunnel, from inspection along the railway, and also over the bare rock cliffs. These cliffs extended for a height of practically 6 000 ft. above the tunnel, which explains how absurd it would be to attempt to make borings in order to get any further information than was apparent from the surface; therefore, the meaning of the phrase previously quoted is entirely different from that placed on it by Mr. Lauchli. It simply indicates that the parties having in mind this method of boring the tunnel had confidence enough in it to know that they could build the tunnel 30 or 40% cheaper than by methods previously used.

Mr. Lauchli further states that his figures of "the cost of driving this bore, with a bottom heading, would be $5 per cu. yd., exclusive of contractor's profit." If this was a fact, all the writer can say is, that Mr. Lauchli's clients must do very well if they can make the proverbial German 1% profit, that is, of course, providing they get any work.

Mr. Keays is under a misapprehension when he states that "no heading was driven in the main tunnel, using the pioneer heading to work from." All the heading in the main tunnel was driven from drifts cut into that tunnel from the pioneer, and all the material from this main heading was carried through the pioneer tunnel, either directly to the outside, or around the shovels to standard-gauge cars back of where the shovels were working in the main tunnel.

Regarding Mr. Thomson's discussion: This is very interesting, and the writer is heartily in accord with his idea of paying homage and doing honor to the remarkable engineers of the past age, and especially to Major Rogers, whom the writer had the pleasure of seeing at a meeting of the Society, in Milwaukee, Wis., some 29 years ago. In this particular detail of the location of the Canadian Pacific Railway, however, Mr. Thomson has a wrong idea. The distance around by the Columbia River, from Beavermouth to Revelstoke, is 163 miles,

* *Engineering News*, February 24th, 1916.

Mr. Sullivan. 96 miles longer than the line located by Major Rogers, and for the traffic carried in 1912 and 1913, when the question of double-tracking and revising the line was being studied, it was seriously proposed to abandon the Rogers Pass route and build by way of the Columbia River, as almost enough could be saved in the cost of operation on this longer line to pay the interest on the construction of the new line, and a great deal more than the difference in the cost of the two lines, had the Company been fortunate enough to build around there in the first place. Nevertheless, the Company is now thankful to Major Rogers, for, had it originally built the line around by way of the Columbia River, the writer is sure that it would never have been abandoned for the present line, which, with the tunnel and heavy business, is more economical, especially in time, than operating the longer distance by way of the Columbia River. The Company should be especially thankful to Mr. Sammy Sykes, the Locating Engineer, who was responsible for the "Loops" on the west slope of the Selkirk Mountains, inasmuch as the original location ran straight down the north bank of the Illicillewaet, where, to this day, signs of some abandoned construction work can be seen, but, in order to save money at the expense of distance, Mr. Sykes put in the "Loops." The result of introducing these "Loops" brought the constructed line into the valley of the Illicillewaet, very close to the West Portal of the Tunnel.

These, the writer thinks, are two instances which illustrate the fact that it is never very safe to be too sure in a criticism. Here are two cases which could be looked on as blunders, as the increased cost of operation, in the one case on account of the heavy grades and the large rise to be overcome and in the other case operating over the longer distance of line without improvement in grades, with a reasonable volume of traffic, would be greater than the interest on the money that was saved in construction, but which really turned out to be a benefit rather than otherwise.

Mr. Davies asks for information regarding the economics of using the pioneer tunnel. To quote further from the writer's report of March 13th, 1913, to the Management:

"This method, of course, is only applicable where the rock will stand without artificial support, at least during the time of construction. Where the material must be artificially supported, then the top heading is the surest, and I think, the best way. The progress of the work by this method, as I said before, depends only on the speed that the pioneer tunnel can be driven. If rock is self-supporting, I see no reason why from 20 to 25 ft. per day could not be made. Placing the cost of driving the small tunnel at $30 per ft., that is the only part of the work that would be rushed under high pressure, and the heading proper can be taken out at least $5 per ft. cheaper than if the work

must be done under pressure, then the bench containing 18 cu. yd. per ft. (neat section) can, on account of there being no interruptions to wait for drilling or cleaning up to put in breast holes or knocking down material in order to get pipes into the heading, at a low estimate, be taken out 75 cents per cu. yd. cheaper, or $13.50 per ft., which would make a saving in excavation of tunnel proper of $18.50 per ft., leaving $11.50 to be taken care of in interest saved account making greater speed. In my report to you of October 22d, 1912, I estimated an annual saving of about $226 000, but all my figures were very conservative, and I took into account only one or two of the larger factors of the extra expense. Mr. Bogue's more accurate figures show a saving of over $370 000. However, his estimate for fuel per h.p. hour was 40% higher than the figure I used, and the price of coal was 17% higher than the price I assumed. His price is more accurate than the one I used, but assuming, for the sake of being conservative, that the average between the two estimates would be approximately correct, that would mean, say $300 000 per year saving, to say nothing of the interest on the $3 000 000 or $4 000 0000 that will be invested in construction, from which we will not be receiving any benefit until the work is completed. Therefore, if this tunnel can be completed one year sooner by using this method, the saving thus made will a great deal more than save the $11.50 additional cost of the pioneer tunnel." Mr. Sullivan.

That these estimates were conservative has been proved by the results, which were better than the estimates. The pioneer tunnel, from the most careful studies of the information at hand, cost about $28 per ft. instead of $30, as estimated. There was a great deal larger difference than 75 cents per cu. yd. between the actual cost of enlarging the tunnel by this method and the estimated cost of enlarging without the use of the pioneer tunnel; and another item, not taken into account in this estimate, was the fact that the pioneer tunnel only had to be driven less than four-fifths of the total distance. This was an indeterminate factor at the time these estimates were made, and the writer purposely omitted referring to it, as it was desired to have the estimates of this untried method conservative in any reports made to the Management.

It has been stated by some that this method is not applicable where there are soft spots in the rock. If the soft rock encountered does not exceed 50% of the total, the writer is confident that this method would still prove more economical than any other which has yet been tried, for the reason that when the soft places are encountered, there is plenty of time to stope out the upper part of the arch and timber it, so that when the steam shovel arrives at those places, there will be no delay whatever, and, instead of having to stope out the entire section by hand, as is necessary in the under-heading method, only about half, or less, of the material in the section requires to be removed in this manner.

Mr. Sullivan.

One word more as to actual costs: To quote further from the report of March 13th, 1913:

"I figure that by this method the pioneer tunnel can be driven for about $30, the main heading for about $40, and that the bench can be taken out for about $54, making a total of $124. There will be incidentals; contractors' profit should not amount to over $20 per ft. Of course, this method of driving the tunnel, working so many drills at one time, will require a larger plant than if only one heading was driven, and that at a slower speed than we contemplate."

The expectations of the Railway Company have been more than realized, as is proved by the speed and the cost of the work. The cost of driving this tunnel through rock, including in this price the cost of driving 19 610 lin. ft. of pioneer tunnel, twelve cross-cuts, each about 40 ft. long, erecting the plant (including freight), the proportionate cost of building about 5 miles of temporary railway tracks, and other overhead charges, plus 10% on all expenditures, will amount to a little less than $5 per cu. yd. for excavation in the tunnel proper.

In conclusion, the writer wishes to say that, in Europe, where it is understood that drill runners get from 90 cents to $1.25 and laborers about 75 cents, the method followed in driving the Loetschberg Tunnel may be both economical and rapid, but all the evidence at hand goes to show that where one pays from $5 to $6 per day for drill runners and from $2.50 to $3.50 per day for laborers, the method adopted is by far the most economical.

Mr. Dennis.

A. C. Dennis,* M. Am. Soc. C. E. (by letter).†—The writer is very grateful to those who have been interested enough to discuss his paper, and wishes to thank them.

In reply to Mr. Lauchli, in his criticism of the Railway Company for its lack of geological data for use of bidders, it would appear that, except for the vicinity of the portals, the Railway Company could not, by boring, or other methods, supply information beyond what was available to the contractors for their inspection of the almost unbroken rock exposure from Rogers Pass to Bear Creek. The kind of rock and its pitch are obvious from inspection along the railway, and from the bare rock cliffs above.

The writer is unable to agree that the bottom heading method of tunneling is "the only method known to-day, which insures absolute success, both under favorable and unfavorable conditions." In good ground, it is obviously an economic question, depending to a large extent on the cost of labor. For heavy ground, such as that near the portals of this tunnel, it is believed that the bottom heading system is unsuitable.

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* Vancouver, B. C., Canada.

† Received by the Secretary, May 9th, 1917.

There was heavy ground near the center of the mountain, in this tunnel, which required timbering. The main heading, from wall-plate level up, was excavated to full section by hand, and timbered by segments from the wall-plate set on rock. When the enlargement and shovel operations reached this point, no delay or damage to the timbering or bench on which the wall-plate was set was caused by blasting or mucking the bench. The work on this section was similar to that of timbering a heading and taking out a bench afterward, except that the muck going out and the timber, pipe lines, etc., in this case, did not interfere with other operations, as would have been the case without a "pioneer" heading. Mr. Dennis.

Mr. Lauchli's figures for bids on this work, and his costs for other tunnels, do not appear to support the bottom heading system as being the cheaper method, even where cheap labor is available.

Mr. Shailer is quite right that 750 boiler h.p. would appear to be inadequate. These boilers, of 750 nominal h.p., however, readily produced 1 000 h.p., by reason of the induced draft. The fuel used was slightly less than the quantity estimated. The fuel estimate in figuring the bid was based on 22 lb. of coal per 1 000 cu. ft. of air compressed to 100 lb. by the gauge.

The compression of air beyond the pressure at which it is to be used, in order to provide for pipe-line losses, is an economic question which was given due consideration. It was calculated that the 8-in. main used was the economic size, and that a larger pipe would not save by reducing frictional losses to compensate for additional first cost, less salvage value, plus labor of putting down and removing. The higher air-pressure requirements noted are the maximum, and only apply for a short period. For a permanent plant, larger pipe would have been justified.

It is doubtful whether modern drills will result in much increase in tunneling speed by the "top heading and bench" system, as suggested by Mr. Keays. The heading drilling may be faster, but the bench drilling and mucking is usually the limiting feature in progress, and hammer drills are inefficient in long down-hole drilling. The writer must disagree with Mr. Keays' statement that the "ventilation between the working face and portal is unimportant", and also that the pressure system is preferable to the exhaust. The exhaust method only requires the mechanical removal of the air from 100 ft. or so of the heading, but the pressure system requires such removal for the entire tunnel length, in order to obtain a clear tunnel. The pressure system tends to mix the fresh air with the smoke and gas, rather than to drive the smoke out bodily. Wooden pipe, in the Northwest at least, solves the ventilating pipe problem, both from an economic and mechanical standpoint.

Mr. Dennis.

Mr. Lavis' words of appreciation are very welcome. The writer regrets that detailed cost figures, which are the real test of any work, have to be withheld to conform with the contractors' policy in this respect, but will go so far as to say that, under like conditions, even if time carried no bonus, he would adopt the pioneer method, unless there was sufficient time to drive the heading, centrally located, entirely through first, and muck the enlargement from one end only, and that never again would he use a top heading in hard rock.

The writer regrets that the report referred to by Mr. Thomson, by the late Virgil G. Bogue, M. Am. Soc. C. E., is not his property, and he is unable to give the Society the pleasure and instruction which was his privilege in studying this masterpiece by his old friend and Chief. Mr. Thomson, as well as many others of the construction days of the Canadian Pacific Railway, will regret that Major Rogers' name, like others who helped in the great work, will soon be forgotten on that railroad. The Rogers Pass Station has been abandoned, and the name of the Rogers Pass Tunnel has been officially changed to Connaught Tunnel.

To Mr. Davis, the writer would say that the idea of a pioneer heading originated in the desire to get away from the congestion, smoke, general confusion, and interference of one operation with another, observed in tunnel driving, and to provide muck in large quantities for handling by shovel. His work in coal mines, with the air course run with the main entry, suggested the pioneer as a means to this desired end. The Connersville blowers were used exclusively for exhausting from the working faces, and would produce 7 lb. below free air pressure. This provided satisfactory ventilation through 12-in. pipe for 1½ miles, at which point a second blower was put in, working as a booster on the same pipe line.

The general wages paid were 40 cents per hour to drill runners, and 35 cents per hour to others. The bonus probably averaged 25% in addition to these rates.

To Mr. Knowles, the writer has to say that the contractor he mentions is the one referred to in the paper as having a sub-contract for labor and explosives in certain headings for a short time, but whose sub-contract was cancelled on account of unsatisfactory work. Mr. White also seems to have been misinformed in crediting this former sub-contractor with the speed made. He and all his organization left the work a year before Mr. White's visit, at which time only a short length of heading had been driven in the rock.

In regard to Mr. Dougherty's point, as to whether this tunnel could not have been built for less cost by other methods, it is difficult to say. The writer does not think so, but would be much interested in a com-

parison of costs of other tunnels with those quoted from Mr. Sullivan's article, due allowance being made for extra cost for long tunnels, and bearing in mind that this tunnel was not in "ideal" or "made-to-order" ground. The West Portal earth section was very difficult, and the rock section was hard to break, as illustrated by the average explosives used in the pioneer heading, that is, 15 lb. of 60% gelatine per cu. yd. Mr. Dennis.

Mr. Dougherty's point, that all excavation should be taken out during construction to allow for lining, is well taken. The extra excavation can be done during construction for probably 10% of the cost of enlarging under traffic. No tunnel is entirely safe until it is lined.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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PAPERS AND DISCUSSIONS

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UNUSUAL COFFER-DAM FOR 1 000-FOOT PIER, NEW YORK CITY

Discussion.*

By F. E. Cudworth, Esq.

Mr. Cudworth.

F. E. Cudworth,† Esq.—The author and others have discussed the larger aspects of this improvement; the speaker will mention some of the smaller things connected with the work, and will take up a few of the problems which arose in carrying out the practical details of the ordering, transporting, handling, setting, and driving, and finally pulling the steel sheet-piling.

Fortunately for the ordering, the Department of Docks and Ferries had complete data, as a wooden pile with a steel shoe had been driven at the corner of each cell or pocket, down to rock. From the rock elevation thus obtained, and assuming a plane surface between the corners, a schedule of piles was made up in lengths varying by 3 in. There were finally more than 3 500 piles in the order. If any one has a similar order to make up, it would be better to use 1 ft. as the variation in length, so as to have to deal with a less number of different lengths.

After giving the matter some thought, it was decided to record the pile data on loose leaves in book form, of a size that could be readily carried in the pocket. The pages contained the necessary information for the rolling mill, the contractor, and the Dock Department, and copies of the books were sent to all parties concerned. As each then had the same information, correspondence and conversation could be readily carried on.

* Discussion of the paper by Charles W. Staniford, M. Am. Soc. C. E., continued from April, 1917, *Proceedings*.

† New York City.

Mr. Cudworth.

The Lackawanna pile, in cross-section, is symmetrical as to the ends, but not the sides, therefore, the names of "right hand" and "left hand" were adopted, on account of the resemblance of the cross-section to the hand, when looked at from the thumb side. All specials were made by splitting the piles and using the halves, so that the end sections must present a certain outline to suit the closing of a pocket with a fixed number of piles—if an odd number, one outline, if an even number, a different outline—so this end section had to be shown also. The number of piles in the side of a pocket was finally decided on as an even number. This made the three wings of a Y-pile show right-handed in plan. The book solved this problem, as it gave the same information to all persons connected with the work, from the mill to the foreman who drove the piles, and the official who inspected them.

The next problem came in the handling. On the first few cars the piles were in no particular order, different cars even became mixed on the railroad lighter and in handling them from the railroad lighter to the contractor's lighter, or to the scows on the work, so that finding a pile of the required length was quite a task, and the loss of time increased the cost. The manner of handling the piles was discussed by the contractor's force and the railroad company, and the speaker went to Buffalo and arranged to have the piles loaded in the reverse order to that in which they were to be driven. The piles were stacked in groups of about eleven pieces each, with a maximum weight of about 11 tons per group. These groups were put on two cars, three groups sideways and two groups high, separated both horizontally and vertically by blocking, so that the groups were unloaded as units in Jersey City. The groups were then transferred in wire rope slings from the railroad lighter to the contractor's lighter or to a scow, from which they could be lifted one at a time by the contractor's lighter. As each single pile was placed, the next one wanted was found on the top of the group, and from one group they went to the next, and so on. This brought order and economy, whereas, at one time it was thought that a large piece of ground would be required for sorting, with a crane or a lighter to do the work.

The next operation was setting the piling. A lighter was brought from Boston for this purpose, but, in sketching it in on the plan at the time the piling was ordered, it was found that the round corner of the coffer would not admit of placing the lighter so that it could do the work. To accomplish this, and also drive the clusters of land piles for tying back the cut-off wall, two booms were put on a floating pile-driver; this was done in order to avoid bringing in a land driver for a few clusters; one boom held the leads and hammer

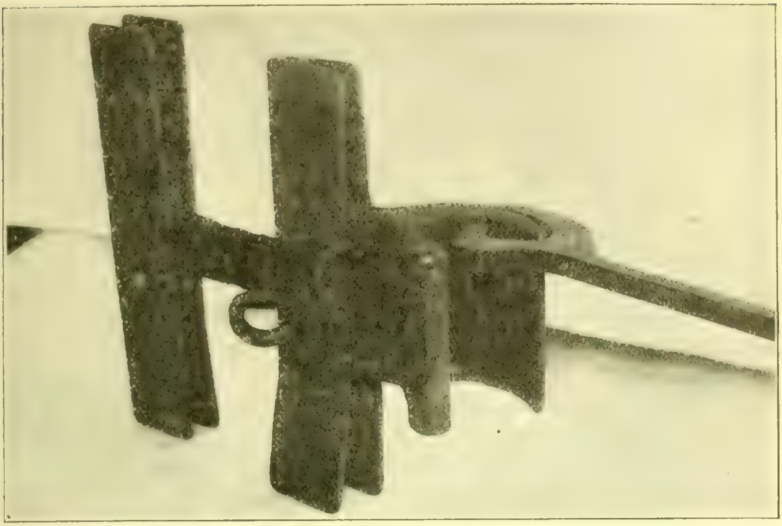


FIG. 37.—GUIDE FOR ENTERING PILE.

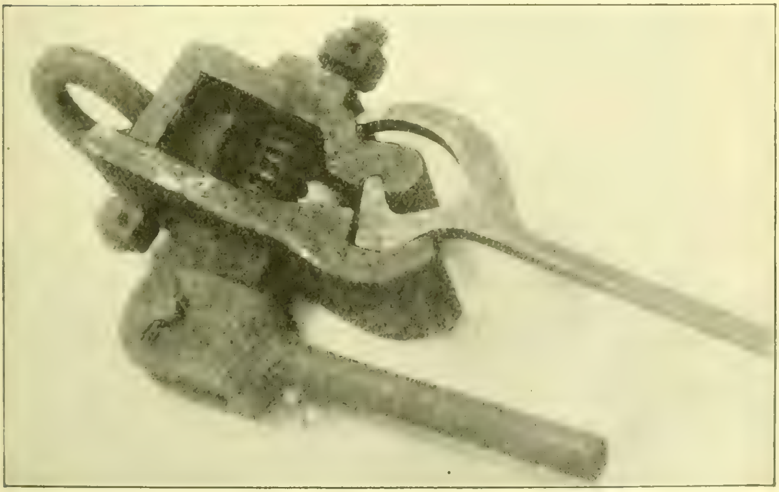


FIG. 38.—SPRING CLAMP FOR ATTACHING TAG LINE TO PILE.

Mr. Cudworth.

and the other handled the pile. In this way the land piles could then be driven and the lighter could be taken into the round corner of the dam "head on," in order to set the steel piles. The driver had little freeboard, but this was remedied by closing the forward hatch to keep the water from flooding the hold. This is the only case known to the speaker where two booms were used on a pile-driver. It was also intended to use these booms later for handling two driving hammers at the same time.

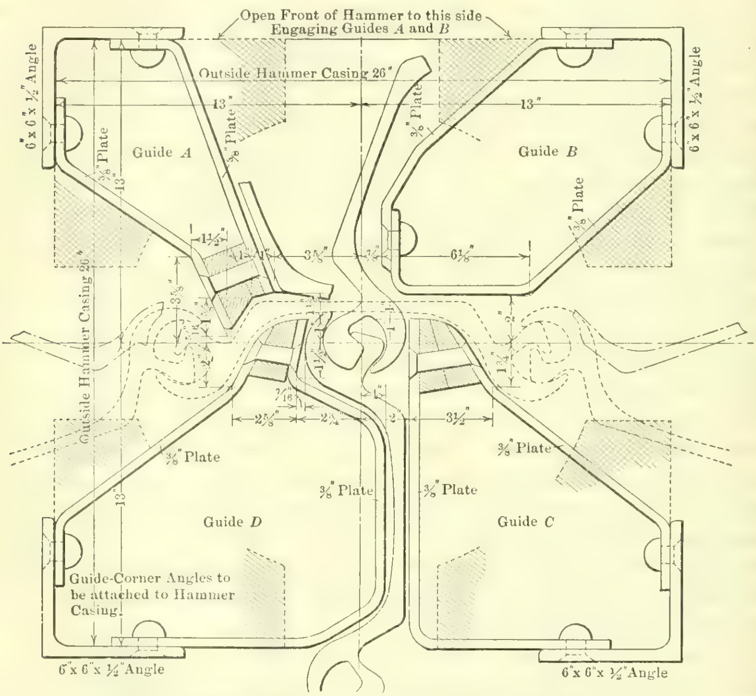
It might be thought that the setting of the piles could be done easily, but, with the end of the previously driven pile 20 ft. above the water, and with a pile from 50 to 70 ft. long swinging in the air from a boom, and the pile-driver or lighter moving with the wind and boat swells, it will be realized that it was quite difficult. It is necessary to enter the Lackawanna pile from the end, and as the clearance is only $\frac{1}{8}$ or $\frac{1}{4}$ in., this is not easily done, and there is a chance that the man guiding the pile may lose a finger or two. Light ladders, with a strong hook in the side, so that they would hang to the top of the pile, were provided, and were better than a boatswain's chair, as the man was on something more nearly stationary. The next thing was to steer the pile into place. Various schemes were suggested, and finally a guide (Fig. 37) was made from a piece of piling about 5 in. long, with a swivel-eye for a safety line, so that it would not be lost if it fell overboard. This guide was held in place on top of the pile with light flat-jaw pieces on each side, and had the "thumb," as that part of the lock on the pilings was called, cut out. The weight of the guide was reduced to 11 lb. by cutting. An operator, skilled in handling a lighter, could lower the pile within the distance of the height of this guide; it could then be pulled into place, and would be held by the "finger" of the guide until lowered. This was a time saver and good accident insurance.

The tag line at first was tied to the lower end of the pile, and two men on two lines held the swing of the pile, and helped the guide man. It took time to put on this rope, and more time when the pile was up in the air, to take it off. A little spring clamp (Fig. 38) was finally made, with a swivel-eye for attaching the tag-lines. The clamp was fastened to the pile with a screw bolt having a lever handle, and this could be put on and taken off in a few seconds. The weight of the clamp was 14 lb.

The best day's work in setting, with one lighter, after the crew had been well broken in, was in one case 98 pieces 70 ft. long, and in another case 129 pieces, from 20 to 25 ft. long, in 8 hours. In the latter instance the average was 16 piles per hour, or a pile in a little less than 4 min. Thus, seconds cut off in different ways were important.

Mr. Cudworth.

After the setting came the driving. The idea was to use the pile-driver boom and the lighter boom, and do away with the clumsy hanging leads. This is difficult, even when the suspension is from a land machine, but, with a floating machine, moving around and bobbing up and down, it is still more difficult. The first trouble was to get the hammer on the pile when up in the air, the next to keep it there, and lastly (but still important), to keep it correctly lined up with the pile, so that the blows would be in the right direction to do the most good.



HORIZONTAL CROSS-SECTION LOOKING DOWNWARD

ATTACHMENT TO HOLD HAMMER
ON ONE, TWO, OR THREE PILES.
(LACKAWANNA ARCH WEB SHAPE.)

FIG. 39.

Finally, an attachment was designed to be placed on the corners of the hammer. This projected like a double clothes-pin. The hammer was the body of the clothes-pin and the attachments were the prongs. This was built in iron and later in wood. Fig. 39 shows the design for the arch-web style of pile; the arrangement for the straight-web pile section was similar. A double set of prongs—making two slots—was used, so that, by placing the piles in one slot, either three or one

could be driven, and by using the other slot, two piles could be covered. The idea was to drive three piles at a time when they drove easily, then two piles, when the driving was harder, and finish by driving one pile at a time, with a follower. This arrangement worked well, and allowed the lifting tackle to be even entirely slacked off, if desirable. In fact, the hammer would work on the pile without being otherwise supported. As shown by Fig. 42, this arrangement was also convenient for taking care of the hammer when not in use, as it saved laying it down on the deck of the lighter. Mr. Cudworth.

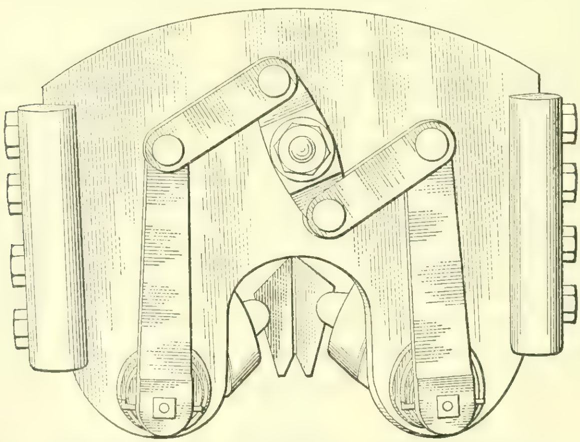
The problem of pulling the piles was a subject of much discussion, both in the field and in the office. On most of the jobs where piles have been pulled heretofore, with the exception of the coffer-dam around the *Maine*, and part of the Black Rock Dam, they had been driven in straight lines, or, if curved, the external was greater than the internal pressure, and they were not necessarily in tension in the locks. Owing to the manner in which the cells at 46th Street were filled, the internal was greater than the external pressure, so that the cells probably took a shape somewhat similar to a barrel, that is, they had swelled out in the middle from the pressure of the materials contained in them, the bottom being held in place by the penetration. The piles to be pulled had been in place for about 2 years, and the earth friction was great. To this was added also the friction due to the tension locks and any additional friction resulting from the effect of the movement of the dam; thus it was difficult to determine the pull necessary to take them out. The locks, however, had been greased when driven.

The problem was how to grip the pile and also be able to let go of it when desired, and how to arrange for the pulling. For taking hold of the pile, the Lackawanna Steel Company was kind enough to lend the contractors a 50-ton and a 150-ton grip, which would engage the web of the pile. (Fig. 40.) These grips were used in pulling the piles in the coffer-dam around the *Maine,* and were of the box form, which allowed the pile, as it was pulled, to come up through the inside of the grip. Where these grips had been used previously, the tops of the piles were at about the same elevation; but in the 46th Street coffer-dam they stood at varying heights, as they had been driven to the rock surface, which was quite uneven. The Lackawanna grip could engage a pile about 8 in. below the piles of either side, but when there was more than that variation it was impossible to apply the grip. To overcome this difficulty the speaker designed a grip of such width that it could go in the 8-in. space between two alternate piles and grab one which was lower. This grip (Fig. 41) had 4 by 16-in. side pieces, closed by a toggle and made to open by a single line operated by the engine runner.

Mr. Cudworth.

This was designed for a 60-ton pull. It was to be used in connection with the No. 1 National Pulling Hammer, as shown in Fig. 43.

This apparatus seemed to be strong enough to do the work. The first piles were pulled in the cut-off with a derrick of about 12 tons capacity. This combination pulled the piles which were shorter than about 25 ft., and then there was difficulty. In this case, due to the small capacity of the derrick, the dead weight of the hammer and grip were too great, in proportion to the striking energy of the hammer, and it was found necessary to do away with the grip, drill holes in the



GRIP USED FOR PULLING THE 75-FOOT SPLICED LENGTHS OF LACKAWANNA STEEL SHEET-PILING FROM THE COFFER-DAM AROUND THE U.S.S. *MAINE*, HAVANA HARBOR.

FIG. 40.

sheeting, and attach the hammer directly thereto. At about this time a new grip (Fig. 45), designed by C. S. Boardman, M. Am. Soc. C. E., of the Lackawanna Steel Sheet-Piling Department, arrived. This was of smaller capacity, of very much better material, and of lighter weight. It was used afterward on the smaller of the two lighters, having a capacity of about 20 tons, and, after some changes, proved very satisfactory. The large grip (mentioned previously) built by the contractor, was used on the 40-ton lighter, and worked well. The most economical and most rapid method of pulling was when the resistance of the pile was within the capacity of the lighter without the hammer.

Mr Cudworth.

PILE PULLING RIG

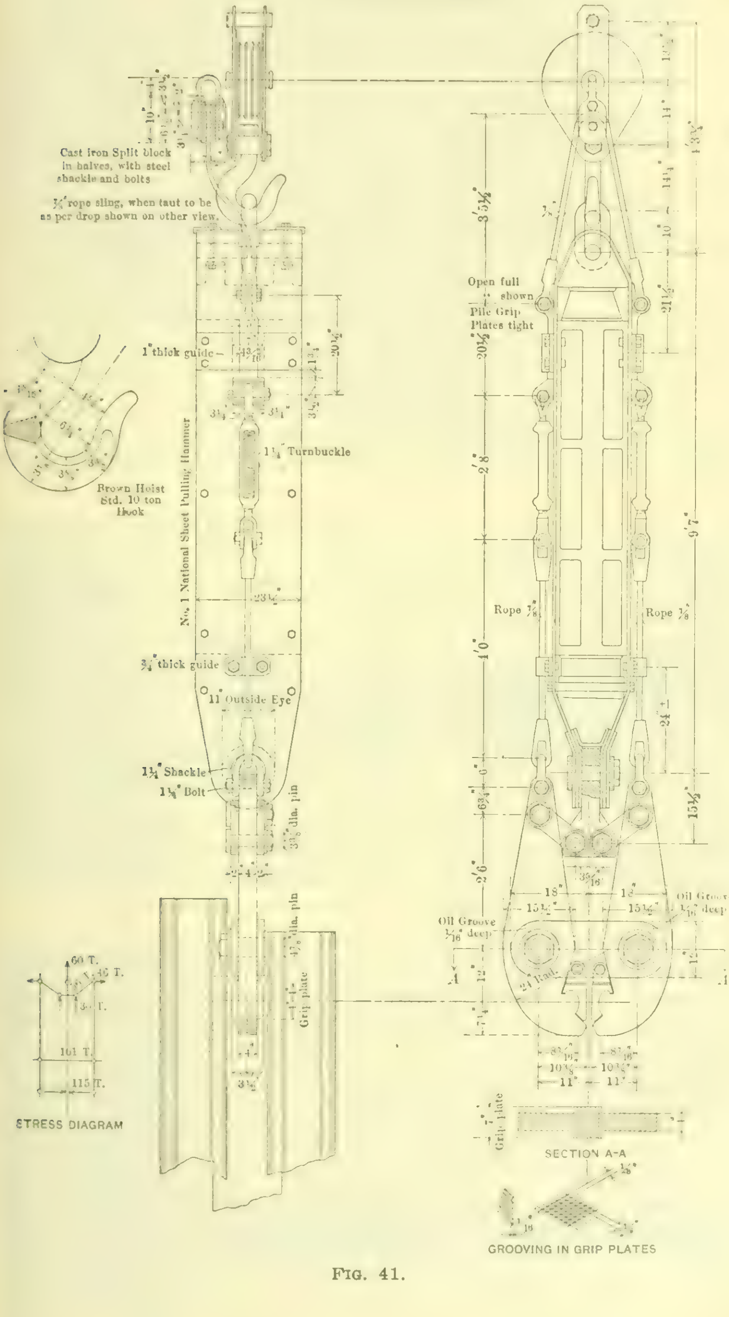


FIG. 41.

Mr. Cudworth.

Mr. Staniford has described some of the ways in which the piles were pulled; the speaker will mention some of the ways in which they were not pulled: In starting the pulling, in the cells where the piles were long, the hardest work, of course, was in breaking the pockets, that is, in pulling the first piece of piling which had tension in the locks on both sides of the pile and was held by the friction of the soil, stone, and other materials. The first combination tried was a lever and two small hammers which were on the job, using as a lever a 24-in., 100-lb. I-beam, 30 ft. long, putting the 35-ton lighter over the grip and the 15-ton crane on the lever. The beam was bent, and failed to start the pile. A heavier lever was made up, using three 24-in., 1 000-lb. I-beams. The crane at the end of the 30-ft. lever lifted about 8 tons, the other end of the lever rested on the top of a Y-pile, 6 ft. from the pile being gripped. This multiplied the lift on the grip to about 30 tons. The lighter pulled about 40 tons directly on the grip, and, in the direct line of the light pull, there were two No. 3 National hammers inverted for pulling. This was estimated at an equivalent of 15 tons pull for each, and made the total lift on the pile about 100 tons. This rig was used because all the materials for it were on the ground; it failed to start the pile.

The two hammers did not work well because there was no way to synchronize the strokes, or blows.

At this time a No. 1 National pulling hammer arrived and was inserted in the line of pull between the lighter and the grip, in place of the two smaller hammers. This arrangement finally started the first pile in the outside row. After this pile had been drawn a good many of the others could be pulled by the lighter with the grip, without the use of the hammer. An air jet (Fig. 44), with a nozzle similar to that used for a hydraulic jet, was put in operation and served to break up the skin friction with the soil. Then hammer trouble started. The band plate, which was of structural material—the best that could be obtained under the market conditions prevailing at that time—did not last very long, and a No. 6 hammer, brought to the job by the McKiernan-Terry Drill Company (Fig. 44), with a band of a different shape was used, as the other bands were expensive and the proper grade of material could not be obtained. The McKiernan-Terry hammer did excellent work.

The pulling was a fight with the piles from start to finish, there being very few days when a good run could be made. The number pulled by one gang ranged from 0 to 40, a fair day's run was 20. The record was made when 66 piles, not exceeding 25 ft. in length, were pulled with a locomotive crane in about 6 hours. This covers the breaking of the outside row.

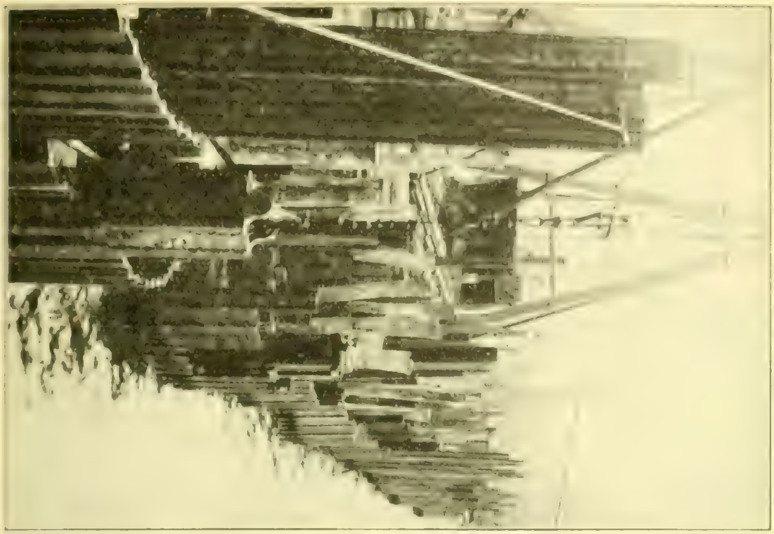


FIG. [illegible]—HAMMER IN OPERATION.

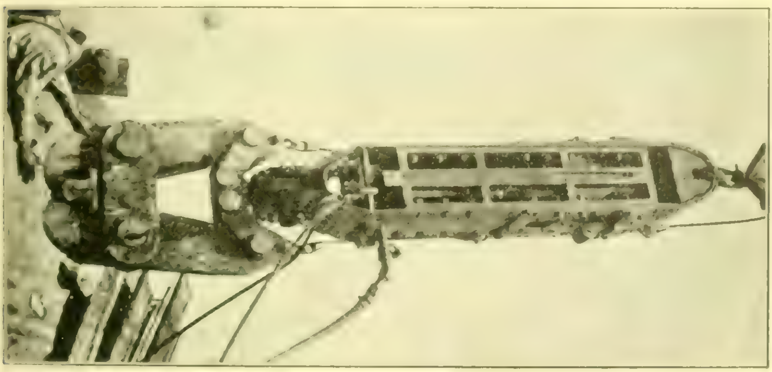


FIG. [illegible]

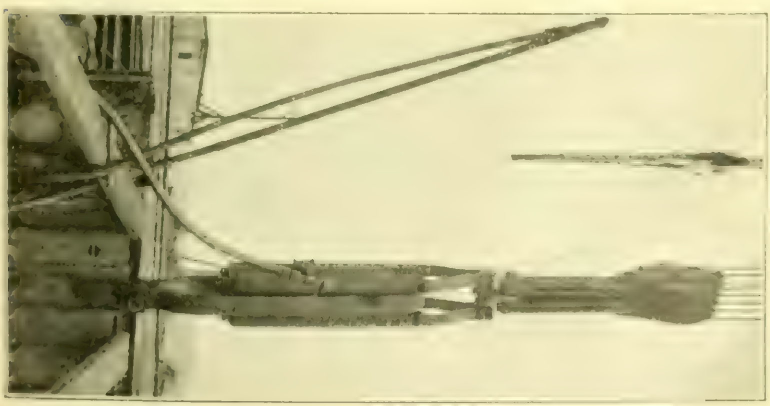


FIG. [illegible]

The breaking of the inside row with an **A**-frame has been described by Mr. Staniford. An **A**-frame was used because, in dredging out the pockets to a depth of about 10 ft., there was not enough of the dam remaining to carry the weight of the cranes. Mr. Cudworth.

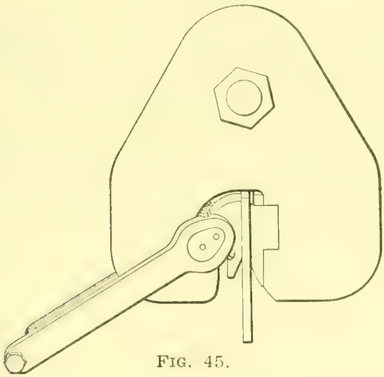


FIG. 45.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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PAPERS AND DISCUSSIONS

TESTS OF CONCRETE SPECIMENS IN SEA WATER AT BOSTON NAVY YARD

Discussion.*

By Messrs. R. J. Wig and Lewis R. Ferguson, and R. E. Bakenhus.†

R. J. Wig‡ and Lewis R. Ferguson,§ Assoc. Members, Am. Soc. C. E. (by letter).‖—The increasing necessity for marine structures, due largely to the improvement of harbor facilities and the military work along our sea coasts, gives an added value at the present time to the paper presented by Mr. Bakenhus. Messrs. Wig and Ferguson.

The points he brings out are largely confirmed by an extensive investigation recently completed by the writers. During this investigation practically all the concrete structures along the Pacific, Atlantic, and Gulf Coasts, as well as a number in Canada and Cuba, were carefully examined. Their present condition was noted, and in most cases a detailed description of the materials and methods used in construction were obtained.

The effect of the quantity of water used in mixing the concrete is rightly emphasized by Mr. Bakenhus, for this factor undoubtedly has a most important bearing on the ability of the structure to resist sea-water action. He states that the specimens made of wet or very wet mixtures are in much better condition than those made of dry concrete, and this is substantiated by the data he presents.

Perhaps it would have been better if the terms "wet" and "very wet" had not been chosen for describing the consistencies used. Ordinarily, in present practice, concrete which would be designated as

* Discussion of the paper by R. E. Bakenhus, M. Am. Soc. C. E., continued from May, 1917, *Proceedings*.

† Author's closure.

‡ Washington, D. C.

§ Philadelphia, Pa.

‖ Received by the Secretary, June 8th, 1917.

Messrs. Wig and Ferguson.

"wet" or "very wet", contains a great deal more water than was used in any of the mixtures from which the specimens were made. Even in the very wet mixtures, the notes in Table 3-*A* indicate that the concrete was tamped. In ordinary practice, very wet or even wet concrete would flow into place, and it would be so soft that tamping would not be necessary, nor could it be compacted in this manner. The writers believe, as a result of their investigation previously referred to, that, with concrete subjected to sea-water action, the best results are invariably obtained when a consistency is used which will permit of light tamping. This light tamping should produce quaking in the mass. If the concrete is mixed too dry, a porous condition will result; and if too much water is used in mixing, a lack of density, together with a chalky condition of the surface, results. In both cases, the concrete is very susceptible to disintegration by sea-water action. For reinforced work, as usually constructed, a slightly wetter mixture is required than what the writers believe to be the best consistency, owing to the necessity of thoroughly embedding the reinforcement.

It is possible, however, that some method may be devised of placing the concrete for reinforced work so that the ideal consistency can be used and, at the same time, secure thorough protection of the embedded steel. The ideal consistency is that which produces concrete of maximum density.

The author indicates that great care was exercised in proportioning the aggregates and in mixing the concrete, the time of mixing, after all the ingredients were placed in the drum, being about 2 min. This undoubtedly had a very beneficial effect on the concrete.

In many of the structures examined by the writers the deterioration in evidence, at least in part, can be attributed to a lack of care in preparing the concrete. More care must be taken in the mixing of the concrete than has too often been the case in the past, if more successful results are to be secured with concrete structures in sea water. In fact, far more care must be taken in every operation connected with constructing concrete which will be subjected to sea-water action than is necessary for land structures.

Only by giving strict attention to minute details, which in land structures would be trivial, can satisfactory results be obtained. Minor defects, which ordinarily would be unimportant, are often the starting points of deterioration which ultimately spreads to sound parts of the structures and cause failure when sea-water action is to be resisted.

Although the information which Mr. Bakenhus presents is of great value for the locality of Boston, care must be taken in drawing too general conclusions from it. There is a great difference in the action of sea water on concrete in different localities due to climatic conditions as well as varying physical actions, such as wave action and abrasion from floating débris and ice.

Messrs. Wig and Ferguson.

The saline content of the water also has a marked effect. The undiluted sea water along our coasts does not vary greatly in the quantity of salt contained, but, where structures are near the mouths of rivers, the sea water is frequently diluted to a considerable extent.

The writers believe that few very general statements can be made on the subject of the effect of sea-water action on concrete, but the peculiarities of each individual locality must be studied before attempting to state the methods to be followed, which will insure permanent concrete structures.

Mr. Bakenhus.

R. E. Bakenhus,* M. Am. Soc. C. E. (by letter).†—The writer has read with interest the various discussions of his paper. Information is offered tending to confirm the results of the Boston tests. The opinion advanced by Mr. Yates that, in setting, a surface is formed on concrete, which will resist successfully the action of sea water on cement, is significant. If a surface coat protects the concrete from the chemical action by sea water, then it becomes very important to know the character of this protective surface, to encourage or bring about its formation, and to protect it from mechanical injury. Mr. Yates cites examples of bridge piers in fresh water showing no deterioration from freezing and thawing and no chemical action, though mechanical injury to the surface exists. This tends to bear out the theory that it is the chemical reaction between the sea water and the concrete that is responsible for most of the deterioration of the latter.

A number of causes are undoubtedly contributory to the deterioration of concrete in tidal sea water. It depends first on the destructive agents existing in the sea water and the atmosphere, and second on the materials of which the concrete is made and the manner in which they are combined. The active possible destructive forces may be listed as follows:

(*a*).—Mechanical injury by floating objects or débris;

(*b*).—Mechanical injury by water, waves, and currents;

(*c*).—Mechanical injury by wind action;

(*d*).—Mechanical injury from freezing and thawing;

(*e*).—Chemical reactions between the sea water and cement;

(*f*).—Chemical reactions between the sea water and the sand or stone of the concrete;

(*g*).—Physico-chemical reactions of the sea water and the concrete, such as the solution of elements of the concrete in sea water, or the formation and deposit, in the pores of the concrete, of crystals occupying greater volume than the original materials, accompanied by minute internal stresses;

(*h*).—A combination of any or all of these destructive effects.

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* Washington, D. C.

† Received by the Secretary, July 30th, 1917.

Mr. Bakenhus.

The elements tending to resist destruction may be listed as follows:

(*a*).—Density of the concrete, that is, reduction of pores to a minimum, accompanied by increased resistance to internal stresses;

(*b*).—Thorough mixture of the ingredients, so as to give a uniform composition of the mass;

(*c*).—Freedom from accidental defects, which may act as starting points of deterioration;

(*d*).—Chemical and physical characteristics of the cement, sand, and stone, making these materials neutral in the tidal range of sea water;

(*e*).—Strengthening of the concrete by a high proportion of cement;

(*f*).—Formation of a protective coat, which is inert in the tidal range of sea water and which, unless mechanically broken, prevents chemical action.

Deterioration is the result of the interaction of the various destructive forces and the resisting elements. In the tests which have been described, the effort was made, by comparing parallel series of specimens, to single out and test the variable elements in the concrete itself, one at a time. The destructive elements of the sea and air might have been similarly segregated, and, in addition, the tests might have been repeated in various climates and in fresh water, as well as in salt water, so that the effect of mechanical injury from floating débris, the effect of wave action, the effect of freezing, the effect of climate and of moisture and air, and, finally, the effect of the chemical and physico-chemical action of the sea water might have been independently determined. In part, this is accomplished by the specimens extending below low tide and above high tide. In the absence of any more elaborate series of tests, the experiences with actual structures in many localities may supplement the data already available, and enable engineers to reach definite conclusions. The writer, therefore, looks forward with interest to the report of the extensive investigations made by Messrs. R. J. Wig and Lewis R. Ferguson, for the Bureau of Standards, in all parts of the country.

In tidal sea water at Boston, the destructive effects, as shown by the tests, occur where the concrete is alternately exposed to the sea water and the air. Exposure to either the sea water or the air alone did not bring about deterioration. This would indicate that it is not simple chemical reactions between sea water and cement that bring about destruction of the concrete, as these could go on below tide level

as well, but that the reactions are dependent on air as well as on water, and are most probably of a physico-chemical nature. Mr. Bakenhus.

It is shown by the tests that certain concretes are not stable in sea water at tide levels. Other concretes are stable in the presence of the forces, chemical and physical, that are active in sea water at tide levels. The dividing line between the two classes is not sharply marked. Knowledge of the action and effect of these forces is just as important to the engineer in designing successful concrete structures in sea water, as knowledge of earth pressure, hydrostatic pressure, or other loads. A very considerable volume of empirical knowledge is now available for the engineer from these tests and from other sources. Under existing conditions, it is apparent that concrete structures in sea water should be planned with care, and only by those who have special knowledge of the subject.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

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THE WATER SUPPLY OF PARKERSBURG, W. VA.

Discussion.*

By Edward Mayo Tolman, Jun. Am. Soc. C. E.

Edward Mayo Tolman,† Jun. Am. Soc. C. E. (by letter).‡—This comprehensive and valuable paper, together with the discussion on the so-called Smith system of filtration which it has evoked has been of special interest to the writer, as that system has been made the subject of several bitter political controversies, into which one side or the other has endeavored to drag him, ever since his appointment as Engineer for the State Health Department, with demands that he either praise the system to the skies, or condemn it as utterly worthless. It was so evident that both sides, for the most part, had so far lost all sight of the engineering principles involved, that a careful report on the system would in no way change the opinion of those interested, but might serve as political capital, that the writer, heretofore, has refrained from an expression of opinion. Nevertheless, the Division of Sanitary Engineering of the State Department of Health of West Virginia has gathered considerable data on the system, some of which may prove of interest. Mr. Tolman.

Beginning in January, 1917, the Department, once a week, has made complete chemical examinations of samples of the Ohio River from directly over the surface of the filters and from a tap in the City Hall. The results of five pairs of samples are given in Table 5.

These results would be of greater value if analyses of water from wells along the river bank were available for comparison. The series of samples were collected primarily for the purpose of showing the percentage of ground-water in the city supply, but the well samples showed

* Discussion of the paper by William M. Hall, M. Am. Soc. C. E., continued from April, 1917, *Proceedings*.

† Charleston, W. Va.

‡ Received by the Secretary, May 17th, 1917.

Mr. Tolman.

TABLE 5.—RESULTS OF CHEMICAL EXAMINATIONS OF WATER, PARKERSBURG WATER-WORKS.

(Results, in parts per million.)

DATE.	JAN. 24, '17.		JAN. 31, '17.		FEB. 7, '17.		FEB. 14, '17.		FEB. 22, '17.	
Source.	River.	Tap.	River.	Tap.	River.	Tap.	River.	Tap.	River.	Tap.
Color	slight.	none.	slight.	none.	slight.	none.	slight.	none.	slight.	none.
Turbidity	very high.	none.	high.	none.	high.	none.	high.	slight.	marked.	none.
Total ash	1055.	130.	180.	95.	320.	90.	75.	90.	190.	110.
Loss on ignition	320.	30.	120.	40.	110.	70.	130.	75.	50.	60.
Free ammonia	0.016	0.008	0.428	0.018	0.046	0.024	0.12	0.066	0.16	0.11
Alb. ammonia	0.91	0.05	0.14	0.04	0.33	0.05	0.13	0.036	0.11	0.04
Nitrites	0.013	0.002	0.010	0.002	0.012	0.002	0.022	0.010	0.014	0.008
Nitrates	0.23	0.31	0.24	0.23	0.39	0.14	0.14	0.23	0.22	0.15
Chlorides	11.5	12.4	10.6	13.8	13.8	12.4	17.5	16.5	25.8	20.7
Oxygen consumed	43.9	0.20	5.5	0.16	11.6	2.7	3.0	1.1	2.7	1.2
Total hardness	79.2	75.7	91.4	73.6	76.5	68.6	72.9	67.5	113.	108.
Alkalinity	20.5	28.	17.5	32.5	13.5	23.5	8.0	8.5	12.5	18.5
Iron	68.	trace.	6.8	0.05	13.2	0.02	2.3	0.	2.2	0.02
Sulphates	....	....	....	....	80.	62.	118.	98.	....	....

such high contamination that the results are of no value in the present discussion. The State Chemist, Mr. Alton A. Cook, in commenting on these results, states: Mr. Tolman.

"That practically no ground-water enters the supply is shown by the figures for ammonia, nitrates, hardness, and alkalinity, since typical pure ground-water would make a greater decrease in the figures for free ammonia than is shown here, and the analytical figures for nitrates, hardness, and alkalinity would tend to be considerably increased under these conditions."

This opinion is interesting as it is generally thought that from 20 to 40% of the water collected in the Smith system at Parkersburg is ground-water. In the writer's opinion, the samples represent very special conditions which can occur only when there has been a rapid rise in the river following a considerable period of low stage, during which the ground-water has had a chance to run out. At Gallipolis, Ohio, and Owensboro, Ky., where somewhat similar systems have been adopted, it is claimed that they are filtering river water, not ground-water, and it may be that such is the case at Parkersburg. However, at Newell, W. Va., where the water supply is obtained from submerged cribs, 4 ft. below the bed of the river, the contrary is certainly true, as shown by the three analyses in Table 6.

TABLE 6.—Results of Chemical Analyses of Water, Newell Water-Works.

(Results, in parts per million.)

Date.	March 19th, 1917.		
Source.	River.	Drilled well.	Tap.
Color	19.	0.	0.
Turbidity	220.	0.	0.
Total ash	180.	195.	185.
Loss on ignition	135.	90.	90.
Free ammonia	0.036	0.006	0.
Alb. ammonia	0.11	0.014	0.012
Nitrites	0.015	0.	0.002
Nitrates	0.18	0.70	1.04
Chlorides	11.8	10.6	6.9
Oxygen consumed	6.0	0.50	0.8
Total hardness	59.	175.	153.
Alkalinity	9.5	130.	95.
Iron	6.6	0.5	0.05
Sulphates	72.	98.	0.

It is evident that, at this town, from 60 to 80% of the water is derived from the underground-water stratum, and is not filtered river water. Similar conditions obtain at Chester, Wellsburg, and other West Virginia towns along the Ohio River, which derive their water supplies from cribs or wells in the river bed. The underground waters

Mr. Tolman. along the Ohio in this State are very hard, and it is probable that sufficient underground water is entering the Parkersburg system to raise the hardness of that supply, during a large portion of the year, above that of a filtered river water, such as would be obtained by rapid sand filtration.

Next, considering the character of the water from the standpoint of turbidity and color: No analyses of the tap water made at the State Laboratory have ever shown either color or turbidity. The highest turbidity recorded by Dr. L. O. Rose, of Parkersburg, who made daily tests throughout 1913 and frequent analyses since then, was 15. This followed a flood in the Miskindum River which empties into the Ohio a few miles above Parkersburg. At the time of the flood, Dr. Rose informs the writer, the results of ten very careful determinations of the turbidity of the Ohio at the filtration plant showed 25 000 parts per million, almost, if not quite, "straight mud." In the fall of 1916, the writer had one of his assistant engineers, Mr. Ellis S. Tisdale, visit some 200 residences and places of business in Parkersburg and inquire whether any turbidity had ever been noticed in the city water and whether the water had always been satisfactory from all other standpoints. Not a single person interviewed had anything but praise for the supply.

That the filtered water has not been highly turbid at times is remarkable, when some of the details of construction of the plant and of operation are considered. With the present design there is no method of telling what proportion of the water is passing through each filter unit. This is particularly dangerous, when it is considered that the pumps suck the water from the collecting manifolds. Thus, if the cover over any one filter should happen to be very thin, this unit would be operated at too great a rate. The water should flow by gravity from the collecting system to the pump-well, and each filter unit should be provided with a rate controller or a recording Venturi. Such devices would furnish data for the intelligent control of the filters. As it is, back-flushing is now conducted with no rhyme or rhythm, as can be seen clearly from an examination of Table 7, which shows a range of from only 1 300 000 gal. pumped per flushing to more than 92 000 000 gal.

The frequency with which the filters are flushed at present depends in no way on the quantity of water pumped or the character of the water. It is the writer's opinion that, in the absence of any controlling devices on the units, daily tests of the turbidity of the river should be made, which, combined with the stage of the river, whether rising or falling, and the quantity of water pumped, would probably serve as a basis for more intelligent filter operation. As it costs approximately $5.80 per million gallons to pump the water, and as between 40 000 and 50 000 gal. are used in a single flush, it will be seen that back-flushing

TABLE 7.

Mr. Tolman.

Month.	1914.		1915.		1916.	
	No. of flushings.	Gallons pumped.	No. of flushings.	Gallons pumped.	No. of flushings.	Gallons pumped.
January.......	10	103 280 000	10	113 030 000	35	122 830 000
February......	25	118 180 000	30	100 600 000	75	97 560 000
March.........	30	95 690 000	30	86 910 000*	25	87 350 000
April..........	0	92 460 000	45	88 630 000	40	113 710 000
May...........	10	95 350 000	60	115 880 000	20	97 970 000
June...........	35	115 170 000	10	99 760 000*	5	92 520 000
July...........	20	119 600 000	15	98 760 000	95	125 570 000
August........	10	105 690 000	40	126 800 000	60	104 300 000
September....	20	140 830 000	75	101 500 000	47†	114 960 000*
October.......	5	91 940 000	10	108 690 000	25	97 880 000
November.....	10	93 770 000	15	87 610 000	10	92 540 000
December.....	15	104 560 000	15	91 500 000	70	115 220 000

* Meter stopped.

† Filters cleaned.

has cost as much as $100 per month, a figure which could be reduced materially if more thought were given to the problem.

As to the purity of the water obtained by the so-called Smith system, at Parkersburg: The analyses made by Dr. Rose, up to the spring of 1916, showed a bacterial efficiency ranging from 95 to 100%, with an average of about 98.8 per cent. In the spring of 1916, the analyses made each month by the State Hygienic Laboratory began to show high bacterial counts and the presence of the colon organism in the filtered water. Dr. Rose then started making check analyses, and obtained similar, though less pronounced, results. To correct this condition, the city, in June, 1916, erected, in the pump-house, a Wallace and Tiernan chlorinating plant. The public has never been advised of this fact for fear that all manner of tastes and odors would be detected immediately. The results of the bacteriological analyses have been more satisfactory since chlorination was begun, but the count is still higher than it should be, and the colon organism is frequently encountered in 10-cu. cm. samples. It is possible that the coli found are entering at the reservoir. The water in the reservoir, owing to the location of the inlet and outlet pipes, has very little circulation, and probably some of it has stood there for many months. Dr. Rose informs the writer that samples taken at a depth have always shown a marked colon content.

In commenting on the results of the chemical analyses (Table 5), Mr. Cook states:

"In general, the purification is not all that can be desired, as shown by the figures for ammonia (free), nitrates, and oxygen consumed in one or more of the last three samples. The city drinking water, on the whole, is of fair quality, but there is room for improvement, before it can be considered a first-class drinking water."

Mr. Tolman.

It may be said, however, that Parkersburg has a satisfactory and safe water supply which is being delivered to the consumer at very low cost.

The writer has frequently heard the so-called Smith system of filtration condemned, on the ground that it cannot produce a safe or satisfactory water. This, he feels, is a mistake. At Owensboro, Ky., Gallipolis, Ohio, Chester and Newell, W. Va., and at other places using systems in many ways similar to that at Parkersburg, the water is satisfactory, both from chemical and bacteriological standpoints. At Chester and Newell, the colon organism has never been encountered, to the writer's knowledge, and yet no disinfectant is used.

It is the writer's opinion that the so-called Smith system of filtration can furnish a satisfactory water at a low operating cost, provided nothing happens to injure the system of under-drains in the river, the problems of proper back-flushing being largely those of design and operation. It would then appear that, other things being equal, the choice of the Smith system over a rapid sand filter hinges on a gambling chance, the engineer betting that nothing will happen to the collecting drains against the high cost of chemicals. If he wins, the profit to the city is large, but, if he loses, the city may pay in life, as well as in dollars and cents.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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PAPERS AND DISCUSSIONS

THE RECONSTRUCTION OF THE STONY RIVER DAM

Discussion.*

By Messrs. Joel D. Justin and Ross M. Riegel.

Joel D. Justin,† M. Am. Soc. C. E. (by letter).‡—This paper teaches a much needed lesson; a lesson not so much needed by the Engineering Profession as by the promoters and owners of projects in connection with which the construction of dams is a feature. Mr. Justin.

If an owner is considering the advisability of constructing a dam, there is one way of no regrets for him to go about the inception of such a project. If he is not already in contact with engineers versed in that particular line of work, he can readily meet them through other owners who have built dams. Having chosen his consulting engineer, the owner should authorize him to make a complete preliminary report on the project, including the economic features. The extent of such preliminary report and investigation should be left largely to the judgment of the engineer, just as the number of visits that a doctor makes his patient should be left to the judgment of the doctor, and not to that of the patient.

Of course, such an engineer should have nothing to sell except service. If the preliminary report is favorable, this ideal owner will say to the engineer:

"Very well, then, we will build this structure as you recommend, and you will consider yourself retained to design the dam and appurtenant work, and to supervise the construction. We will expect

* Discussion of the paper by F. W. Scheidenhelm, M. Am. Soc. C. E., continued from May, 1917, *Proceedings*.

† Chippewa Falls, Wis.

‡ Received by the Secretary, June 25th, 1917.

Mr. Justin.

you to have on the ground at all times competent assistants to see that your orders are executed. In this matter, Mr. Engineer, we put ourselves entirely in your hands."

It is true that the best of engineers sometimes make mistakes. So do doctors and lawyers. The difference is that the two last named professions have an excellent opportunity of "getting away with it", whereas the mistakes of the engineer stare the whole world in the face, for they can be seen and photographed. It is probably for this reason that the business man, who accepts as infallible the advice of his lawyer or doctor, questions and often ignores the advice of his engineer. It is certain, however, that if a larger proportion of owners were to follow the course of action just outlined, there would be fewer failures charged to the Engineering Profession.

In the case of the Stony River Dam, a competent consulting engineer was retained, apparently merely to investigate the feasibility of the site. His connection with the project ceased when the contract for the construction was let, and he apparently had nothing to do with the design. Had he also been retained to design and supervise the construction of the dam, it is improbable that Mr. Scheidenhelm would now be writing about its reconstruction.

Hollow dams of the type used at Stony River have manifest advantages over other types for the conditions existing at the chosen site. In many such dams insufficient attention has been given to foundation problems. It would almost appear that it has been quite customary to use what may be designated as stock designs without much regard for local conditions, and without very thorough sub-surface investigations.

The paper proves that the author went into this matter in a most thorough and complete manner. He realized that a reconstruction of this nature required as much if not more engineering investigation than an entirely new project. In spite of his convincing hydrological study, Mr. Scheidenhelm's provision for a spillway capacity of 1 840 sec-ft. per sq. mile seems rather high. Much of this, it is true, is obtained by merely allowing spilling over the bulkhead section or "Intermediate Spillway." It appears probable, from the author's study, that the duration of a flood when the capacity of the original spillway would be exceeded would be brief. Consequently, it would seem that the author might have saved the cost of the down-stream apron on the new spillway. The abutments at the east and west ends of the masonry structure might have been raised several feet to prevent water flowing over the earth, and then, when the capacity of the old spillway section was exceeded, discharge would take place over practically the entire width of the dam. It would have been necessary, of course, to extend the tumbling hearth for the entire distance. It

would probably also have been desirable to provide an additional apron to protect the soil just down stream from the dam. Mr. Justin.

The author shows most conclusively that the resistance to sliding of the structure as originally built was insufficient. This condition is no doubt quite often the case with existing hollow reinforced concrete dams built on soil foundations. In the design of such structures there has often been far too little attention given to making adequate provision against sliding. The methods adopted by the author for increasing the resistance to sliding are most ingenious and effective. At the same time their execution must have been expensive.

After investigation the author decided that the safe soil pressure had already been reached, and therefore he did not consider it safe to obtain the desired additional resistance to sliding by filling in with earth on the floor of the dam. This objection might have been overcome by extending the reinforced concrete floor or footings down stream from the buttresses until the desired maximum soil pressure was attained. To take the reaction from the soil pressure it would have been necessary to cantilever the buttresses out in a down-stream direction over the additional floor or apron. This cantilever would probably take the form of a triangular addition to the buttresses at their down-stream ends. The earth filling, to produce the necessary weight, would be placed inside the dam, and the bulk of it as far up stream as possible, thus obtaining a lower maximum soil pressure per unit of area at the toe than if the fill were evenly distributed.

The two problems of securing proper resistance to sliding and adequate cut-off should be handled as one, and this the author has done. The construction of the cantilever heel, the underpinning, and the extension of the original cut-off were clearly very expensive. This work, aside from the mere grouting of leaky joints and of voids along the cut-off, would be omitted in the method the writer has in mind. In the case of a dam on a soil foundation, the construction of an impervious cut-off to rock is frequently out of the question. The problem is more generally one of introducing sufficient frictional resistance to the flow of water under the dam, so that the velocity of the water issuing from the down-stream toe is insufficient to move the foundation soil. In fact, the desirability, in the case of a yielding soil, of a deep cut-off at the heel to rock is open to question, as it produces a tendency to increase the maximum soil pressure at the toe. The necessary frictional resistance to the flow of water under the dam is obtained by increasing the distance through which the water must flow before it can escape at the down-stream toe. This may be obtained by (1) increasing the depth of the cut-off, (2) by increasing the width of the base, (3) by a combination of these two methods.

Mr. Justin.

The additional down-stream floor or apron, suggested by the writer as a method of obtaining decreased soil pressure, would, by increasing the frictional resistance, decrease the velocity of the water flowing through the soil. This, however, would probably not be sufficient. Therefore, the writer would use an up-stream apron. The thickness of this layer need be only 6 in. or indeed it might consist of a few layers of "Gunite" on a light-mesh reinforcement, there being slight stresses to be resisted. For the typical section, the width of this apron should probably be about 100 ft., as it would not be wise to place great dependence on the original cut-off. At the junction of the apron and the heel there should be a special flexible joint which would remain tight even if there should be a slight movement in the dam. A small fill of selected clay should also be placed at this point. The alternative for providing against seepage and sliding here suggested is shown in Fig. 39, the dimensions being merely illustrative.

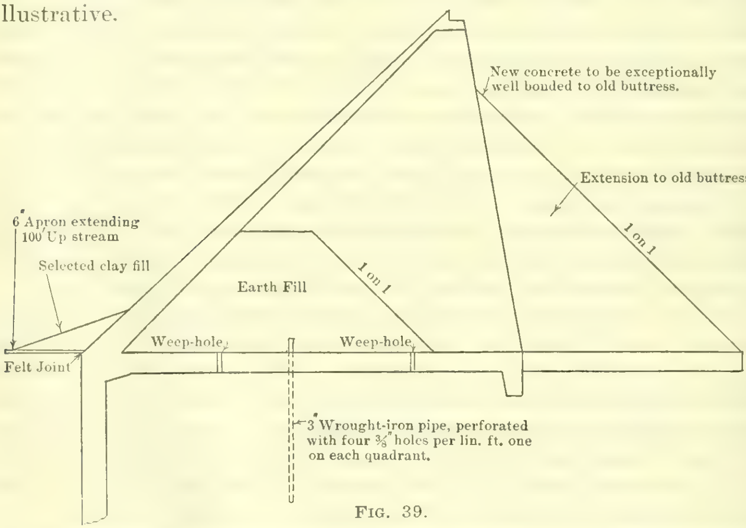

FIG. 39.

In outlining this method of increasing the resistance to sliding and to seepage, the writer realizes that it was doubtless considered by Mr. Sheidenhelm. The only reason for suggesting it is, that, from the author's description, it would appear that it would have been cheaper than the equally efficacious one adopted.

Mr. Riegel.

Ross M. Riegel,* Assoc. M. Am. Soc. C. E. (by letter).†—The writer desires to commend the practice of using a hypothetical storm with a graph of the run-off, in determining spillway characteristics. His observation has been that, in the design of most dams, this deter-

* Dayton, Ohio.

† Received by the Secretary, July 6th, 1917.

mination has been largely a matter of conjecture, as is attested by numerous failures. The author's practice is similar to that of the Miami Conservancy District in the design of flood-control dams for the Miami Valley in Ohio. In the latter case, recorded gauge heights enabled a graph of the run-off to be constructed for the great flood of March, 1913. From this, by the addition of a margin of safety of about 40%, another graph was developed for the determination of the spillway and outlet capacities for the dams. Mr. Riegel.

Although the spillway capacity of the Stony River Dam is high as compared with the usual practice, the circumstances of high altitude, heavy rainfall, and small drainage area, and the fact that the dam as reconstructed must not fail, combine to justify an attitude of great conservatism.

The essentials of dam construction, aside from spillway capacity, are a tight face to retain the water, a stable structure to maintain the integrity of the face, and satisfactory drainage facilities to carry away any water which may pass the face or the foundation. In all these essentials the original dam appears to have been deficient. The tight face was wanting by reason of an insufficient and faulty cut-off; the drainage provision was such that it was subject to interference by freezing during a great part of the year; and the resistance to sliding seems to have been markedly deficient.

In the reconstruction, these faults have been remedied with a degree of thoroughness not often seen. The improvements at the heel, as well as the various extensions and underpinning of the cut-off, have corrected the faults in the original face. In his discussion, Mr. Rutenberg seems to have overlooked this point entirely. Had nothing been done to improve the cut-off, the drainage provisions of the reconstructed dam might indeed have been insufficient, and Mr. Rutenberg's points would have been well taken. The fact that such leakage as did occur after the dam was refilled was readily checked, or checked itself after a short time, is ample testimony to the effectiveness of the measures taken.

The provision made to protect the drainage outlets from freezing, namely, housing in the base of the dam, savors of the heroic, but seems to be justifiable. The author was repairing an existing structure, and to have developed an ideal system of sub-drainage with protected outlets below the dam was evidently prohibitive as to cost. Under the system adopted, any seepage water will find its readiest avenue of escape through the weep-holes or pipes. Its escape immediately under the footing will be practically prevented by the new "toe" wall. Moreover, after passing the improved cut-off, the motion of seepage water through any strata below this wall will be so slow that no erosion need be feared.

The provision of increased resistance against sliding seems to have been necessary and to have been well worked out. The writer fails to

Mr. Riegel. see any better method of remedying this defect, and feels that the device is worthy of imitation. Although the criticism has been made that the heel- and toe-walls are too light to carry the entire horizontal thrust, it seems to the writer that such a requirement would be excessive. Their function is to add to the resistance normally offered along the top surface of the foundation itself, and, therefore, their strength is doubtless sufficient.

On the whole, the writer has been impressed by the care and thoroughness of the author's methods. He wholly fails to comprehend Mr. Rutenberg's criticism, which seems to have been based on rather hasty consideration. For instance, his remark that the dam is less safe now than before reconstruction seems particularly unsound, inasmuch as the original dam failed after being in ordinary service for 65 days, and the reconstructed one has now been in service for 2 years.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

PAPERS AND DISCUSSIONS

CEMENT JOINTS FOR CAST-IRON WATER MAINS

Discussion.*

By Messrs. H. B. Lynch, Edward R. Bowen, and George W. Pracy.

H. B. Lynch,† Esq. (by letter).‡—This method of caulking joints in cast-iron pipe deserves a wider use, not only on account of its low cost, but because it makes a better joint, at least where no considerable movement of the pipe is expected after it is laid. Mr. Lynch.

Cement joints have been adopted as standard at Glendale, Cal., and have been used on 10 miles of cast-iron pipe laid in the past 2 years. This joint was not adopted until after investigation had shown that it is being used with perfect satisfaction in various Southern California localities, and tests had been made to show its effectiveness. The results obtained at Glendale have been similar to those described in the paper. No joint yet placed has shown permanent leakage, and, in only four, has the presence of dampness been detected after the pressure was turned on. No joint has required recaulking.

The strength of this joint has been shown at Glendale by several incidents of various kinds. As a matter of fact, it is customary for the pipe crew to take the greatest liberty with the pipe after the joints are completed, as experience has shown that a joint properly made will not be started by quite severe treatment. A 3-mile pumping main of 20-in. cast-iron pipe recently completed was entirely laid up with cement, at an estimated saving of more than \$3 000. On this line, when a nipple was to be caulked into a cross, it was customary

* Discussion of the paper by Clark H. Shaw, Assoc. M. Am. Soc. C. E., continued from May, 1917, *Proceedings*.

† Mgr., Public Service Dept., Glendale, Cal.

‡ Received by the Secretary, May 23d, 1917.

Mr. Lynch. for the men to stand the cross on end on the surface of the ground and set the nipple into the top bell, and caulk up on the surface. The nipple and cross, weighing 3 000 lb. were then picked up with the chain blocks and lowered into the trench, without waiting for the set to occur. No joint treated in this way showed the slightest leakage or dampness under a pressure of 90 lb. On one occasion, a 20-in. cross had caulked into it a plug, a reducer, and a 20-in. nipple, 10 ft. long. This whole assembly, weighing about 4 000 lb., was lowered into place as soon as caulked, and is at present operating under a pressure of 90 lb. per sq. in. without seepage.

This 20-in. line was connected to a 16-in. riveted steel line. For this purpose the 16-in. line was taken out of service at 8 P. M., and put back at 4.30 A. M. Joints were given 4½ hours to set. Out of eight joints thus treated, one remained damp for one day. One joint, where the 16-in. riveted line was caulked into a cast-iron bell, dripped for two days and then took up tight. This joint showed a face of about 1¼ in. between the pipe and the bell. No other joint on this job showed any initial seepage, and all are tight now.

The method of making a joint in Glendale is much the same as that described in the paper. After yarning, the joint is rammed full of slightly damp cement. Great care is taken to caulk this first ring of cement thoroughly, as this is the greatest factor in a successful joint. No bead is now used, as no difference in results is noted when this is omitted. After caulking, the joints are covered with earth for protection against the sun.

The tools used are slightly wider than those used for caulking with lead.

In spite of the strength of the joint, its removal is surprisingly easy. At different times, 20-in. plugs have been taken out, and there has not been the slightest difficulty where the method described in the paper was used. To caulk a joint of this size the writer allows about one hour's time of one man.

Mr. Bowen. EDWARD R. BOWEN,* ASSOC. M. AM. SOC. C. E. (by letter).†—The writer has read this paper with keen interest. It gives in admirable detail the results of an extended experience in handling the cement joint in cast-iron pipe construction. The advantages of this type of joint have not been generally realized among water-works engineers. It is not only less expensive than the lead joint, but, in some ways, distinctly better.

As protection against electrolysis it is almost a perfect insulator. Tests made at the Long Beach Water Department yards showed the

* Los Angeles, Cal.

† Received by the Secretary, June 11th, 1917.

resistance of cement joints on an 8-in. line to be nearly 19 ohms, or more than three times the resistance of the ordinary lead joint. Because of the high resistance, under ordinary electrolytic conditions, extremely small currents could travel along the pipe line. Tests were made in Long Beach in the vicinity of an electric railway sub-station to determine the quantity of current carried by the water-pipe lines. In this particular location there were cast-iron lines with both lead and cement joints. The position of the pipe lines with respect to their susceptibility to electrolytic action was practically the same. An appreciable flow of current was discovered in the pipe lines with the lead joints, and a very much smaller flow was noted in the line with the cement joints. In the latter instance, however, the same drop in potential could be obtained by inserting the connections in the soil at the same distance apart as they were applied to the pipe lines. Mr. Bowen.

In the writer's opinion, the success or failure of the cement joint depends solely on the method used in its construction. It is extremely important to have as little moisture in the cement as possible. The mixture described in the paper has proved to be entirely satisfactory, and care should be taken to avoid using more moisture than is there recommended. If more water is used the cement in setting shrinks away from the pipe, resulting in a leaky connection.

Tables 6 and 7 have been prepared to illustrate the relative costs of the two types of joints. Table 6 is based on the data in the paper. Table 7 is based on the cost data assembled by the Los Angeles City Water Department.

TABLE 6.—COST OF CEMENT JOINTS.

Diameter of pipe, in inches.	JUTE.		CEMENT.		LABOR.		Total cost of joint.
	Pounds per joint.	Cost at 6 cents per pound.	No. of joints per sack.	Cost at $1.00 per sack.	No. of joints per 8-hour day.	Cost at $2.75 per day.	
4	0.14	$0.008	24	$0.042	50	$0.055	$0.105
6	0.19	0.011	18	0.056	42	0.065	0.132
8	0.24	0.014	14	0.071	34	0.081	0.167
10	0.43	0.026	11	0.091	28	0.098	0.215
12	0.51	0.031	8	0.125	24	0.115	0.270
14	0.58	0.035	7	0.143	20	0.138	0.315
16	0.66	0.040	6	0.167	17	0.162	0.368
18	0.73	0.044	5	0.200	14	0.196	0.440
20	0.80	0.048	4	0.250	11	0.250	0.548
24	0.95	0.057	3	0.333	7	0.393	0.783

Mr. Bowen.

TABLE 7.—COST OF LEAD JOINTS.

Diameter of pipe, in inches.	Lead.		Yarn.		Fuel at 0.6 cent per pound of lead.	Labor cost, based on $2.75 per man per day.	Total cost of joint.
	Pounds per joint.	Cost at 6 cents per pound	Pounds per joint.	Cost at 6 cents per pound.			
4	7	$0.420	0.2	$0.012	$0.042	$0.110	$0.584
6	9	0.540	0.4	0.024	0.054	0.168	0.786
8	14	0.840	0.5	0.030	0.084	0.216	1.170
10	16	0.960	0.6	0.036	0.096	0.230	1.322
12	20	1.200	0.8	0.048	0.120	0.264	1.632
14	24	1.440	0.85	0.051	0.144	0.310	1.945
16	29	1.740	1.0	0.060	0.174	0.324	2.298
18	31	1.860	1.1	0.066	0.186	0.360	2.472
20	35	2.100	1.3	0.078	0.210	0.432	2.820
24	40	2.400	1.5	0.090	0.240	0.492	3.222

Mr. Pracy.

GEORGE W. PRACY,* ASSOC. M. AM. SOC. C. E. (by letter).†—Several months ago the Spring Valley Water Company laid 4 730 ft. of 4-in., 900 ft. of 6-in., and 827 ft. of 8-in. cast-iron bell-and-spigot pipe, with cement joints.

The cement was mixed by one man who handed it around to the two or three men making the joints. No caulking was done on the joints, the cement being tamped by hand with a caulking iron. After the joint was full, a bead was put on around the face of the bell. Neat cement was used. Just as little water as possible was added in mixing. When ready for use, the wet cement was still dry enough to crumble when handled. The first few joints were filled with a wet mix, with the result that, in these joints, the cement shrunk away from the iron, and all the joints had to be remade. As an experiment, one of the 8-in. joints was caulked hard, using an extra dry cement. This gave a good joint, but not better than the others, and it took twice as long to make.

Water was turned into the pipes 48 hours after the last joint was made. For the first day nearly every joint leaked. After that, they took up rapidly and, at the end of 1 week, all were tight.

The pipe was laid in five sections of about 1 300 ft. each. All but one section was tested by measuring the quantity pumped into the section during a given period. Tests were made after the pipe had been under ordinary working pressure for about 1 week to 10 days. For the first section laid, this leakage was $\frac{1}{3}$ gal. per lin. ft. of pipe joint per 24 hours. The second leaked so slowly that the leakage could not be measured. The last two sections laid were absolutely tight. The lines were pumped to pressure and left standing, in one

* San Francisco, Cal.

† Received by the Secretary, July 30th, 1917.

case, for ½ hour and in the other case for 2½ hours, without any drop in pressure. The 6-in. pipe section could not be tested, as it was necessary to put it into use immediately. Mr. Pracy.

The quantity of cement used was as follows:

No. of joints.	Size.	Quantity of cement, in pounds.
187	4-in.	750
2	5-in.	
12	8-in.	
78	6-in	360
168	4-in.	1 715
86	8-in.	

These figures include all waste. On an average the 4-in. joints took about 4 lb., the 6-in. joints 5 lb., and the 8-in. joints from 7 to 8 lb. These figures are rough approximations, based on the theoretical quantity necessary for a joint of each size.

The fire hydrants on the line were set with lead joints, as it was feared that the vibration of the hydrants, when in use, would break the cement joints.

The lines have given entire satisfaction.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

MULTIPLE-ARCH DAMS ON RUSH CREEK, CALIFORNIA

Discussion.*

By Messrs. F. W. Scheidenhelm, Edward Wegmann, Walter J. Douglas, Edwin Duryea, L. H. Nishkian, Gardner S. Williams, and George W. Howson.

F. W. Scheidenhelm,† M. Am. Soc. C. E.—The author has presented an interesting and detailed analysis of certain features of multiple-arch dam design in such a manner as to deserve the appreciation of the members of the Profession interested in dam design and construction. In addition to the general analysis, he has given an outline of the special features pertaining to the design and construction of the two particular multiple-arch dams at Gem Lake and Agnew Lake. Essentially, however, the paper is general in its character and, correspondingly, this discussion will also be mainly of a general nature. Mr. Scheidenhelm.

The multiple-arch dam, like the flat-deck dam, belongs to what one might call the family of hollow dams. The multiple-arch type differs from the flat-deck type (of which the Ambursen dam is the most familiar) primarily in the method of transferring the water load to the supporting buttresses. Both types involve relatively high unit costs for the concrete work required. Accordingly, they find greatest favor in situations where it is proposed to build dams of concrete, but where the cost of cement is high. The dams at Gem and Agnew Lakes are cases in point, for there the average cost of cement delivered at the site of the work is reported to have been \$7.50 per bbl. Apparently, only rock-fill dams, but not solid masonry dams, were considered as alternatives for multiple-arch dams for the sites mentioned. It seems evident, however, that, had solid masonry dams been built, the high

* Discussion of the paper by L. R. Jorgensen, M. Am. Soc. C. E., continued from May, 1917, *Proceedings*.

† New York City.

Mr. Scheidenhelm.

cost of cement would have more than offset the relatively high cost of the form work required for the multiple-arch dams.

As to hollow dams, there appear to be two principal points which require careful attention, namely, the protection of reinforcing steel and the tendency to skimp the sizes of the structural members. In the first respect, the multiple-arch type has a decided advantage over the flat-deck type, in that it requires a much smaller quantity of steel. Consequently, there is so much less steel which might be placed in danger of corrosion, and it is so much the easier to protect the steel. Again, the multiple-arch dam permits the use of longer spans, say, from 20 to 50 ft. between centers of buttresses, as compared with corresponding spacings for the flat-deck type of from 15 to 30 ft.

On the other hand, the cost of form work for the multiple-arch dam is somewhat greater than that for the flat-deck type, and, what is more important, the multiple-arch dam is more susceptible to difficulties and damage in case of settlement of the footings. The arches cannot withstand as much racking as flat-deck slabs. The author points out that he provides struts to take care of the unbalanced arch thrust which would result if one or more arches or bays were to fail. Of course, a dam is intended not to fail, and, hence, one is concerned more with precautions against failure than with measures to be taken in case of failure. Even from the latter point of view, however, the value of the struts would be problematical were they, in case of failure of the adjacent arches, to be subject to the impact of floating ice, tree trunks, or other débris. Moreover, it would seem that the struts might become overloaded due to blocking the passage of such débris, and thus induce failure in lateral flexure.

After all, it appears to the speaker that, in the case of the multiple-arch dam, one must insist on unyielding foundations. Fortunately, there seems to be no deficiency in bearing value on the part of the foundations of the dams on Rush Creek.

It is not surprising to find that the use of the multiple-arch dam dates far back of that of the flat-deck type, at least in so far as the construction of the latter in reinforced concrete is concerned. The most conspicuous example is that of the Meer Alum multiple-arch dam, built in India a century or more ago. This dam, referred to by Mr. Flinn, has a height of about 45 ft., and involves astounding span lengths, varying from 70 to 147 ft. The arches are plain, and are built of brick in lime mortar. New South Wales boasts of a dam, the Belubula, in which the spandrels of the arches are filled in with masonry, thus making impossible radial action of the water pressure.

In the United States there are, perhaps, a dozen multiple-arch dams, the most familiar of which heretofore have presumably been those built under the design and direction of John S. Eastwood, M. Am. Soc. C. E. The Eastwood multiple-arch dams seem to differ from those

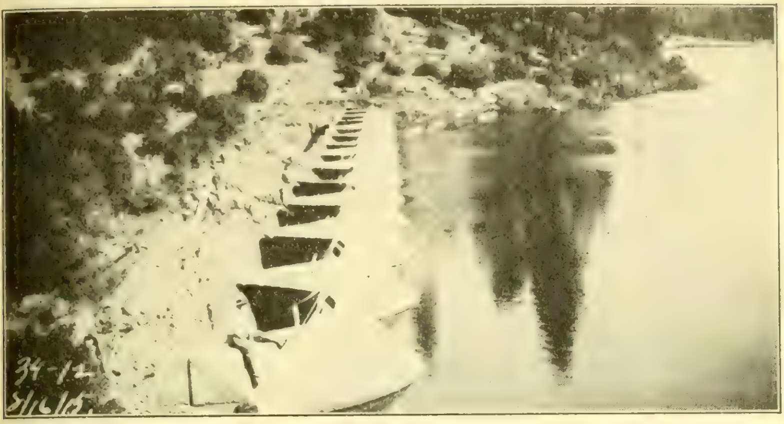

Fig. 19.—Los Verjels Dam, Near Oroville, Cal.

Fig. 20.—Down-stream View of Los Verjels Dam.

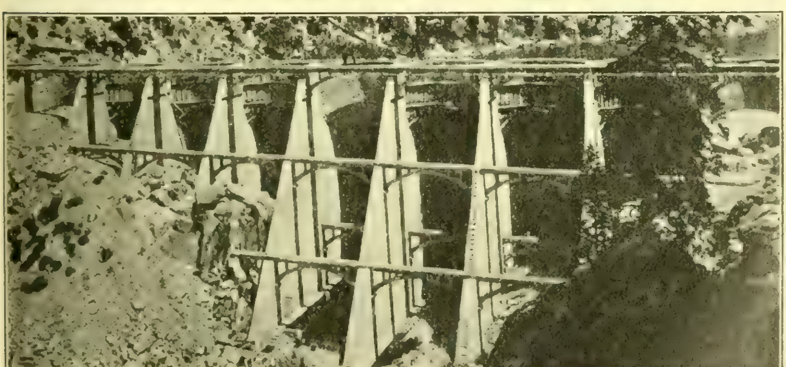

Fig. 21.—Down-stream View of New Big Bear Valley Dam.

designed by the author essentially as regards the treatment of the arches near the crest. Mr. Jorgensen uses an elliptical, rather than a circular, form for, say, the upper 15 ft. of each arch or panel, whereas Mr. Eastwood apparently prefers to retain circular arches, even near the crest, and, to this end, has made the arch axes vertical for the upper portions of the panels of certain of his dams. Of this design are the dams at Hume Lake and Bear Valley, California. Mr. Scheidenhelm

The speaker has not had the privilege of visiting either of the dams described by the author, but has had the opportunity of inspecting the 60-ft. high multiple-arch dam of the Los Verjels Land and Water Company, near Oroville, Cal. This dam is illustrated in Figs. 19 and 20. The former is a view along the spillway crest. Incidentally, it emphasizes the lack of adequate spillway capacity. This is shown by the erosion which is apparent at the opposite bank.

Fig. 20, a view from down stream, shows the buttresses and bracing to be relatively slim, to such an extent, in fact, as to afford to the eye no impression of stability.

The Bear Valley Dam, Fig. 21, makes quite a contrary impression, and, though the product of the same designer, governed as he apparently was by the financial ability of the respective clients, is of pleasing appearance and proportions. Even in this case, the speaker would prefer a system of bracing between buttresses consisting of relatively deep (vertically) beams or walls. Such deeper beams would certainly afford greater stiffness, and, moreover, would tend toward a better distribution of the load on the footings. In other words, with the buttresses well braced, any deflection or settlement of the different footings would, of necessity, be practically equal, and hence the firmer, less yielding, foundation material would, as is desirable, take more load per unit of area than the less firm portions. The last-mentioned advantage of the deeper bracing beams is, to be sure, of little value in foundations where the deflection under load is inappreciable.

Turning now to the details of the paper, the speaker is inclined to think that the multiple-arch dam may properly be applied even more widely than the author claims, for he, by implication, limits its utility to locations where "the foundation is solid rock." Nevertheless, the multiple-arch dam is not entirely free from the ills of other dams. More particularly the speaker cannot agree with the general statement of the author that "there is no hydrostatic uplift to amount to anything acting on a dam of this type." Although not obsessed with such a fear of uplift pressure as is shown by some engineers, the speaker insists that the possibility, and often the certainty, of uplift pressure must be recognized.

In the absence of more definite information, the allowance made for uplift pressure must be, of necessity, a matter of judgment. Certainly, however, in the case of a foundation material which, where

Mr. Scheidenhelm.

unbroken, is impervious, but contains approximately horizontal laminations, it must be admitted that uplift pressure may exist in such laminations or seams. Under such circumstances, the effect of the uplift pressure would be practically the same as if the footings of the buttressed hollow dam were continuous, with uplift pressure existing in the plane of contact of the footings with the surface of the foundation material. Manifestly, with rock of the characteristics previously suggested, a buttressed hollow dam, such as the multiple-arch dam, has no advantage over the solid masonry dam, except to the extent that, by reason of the details of construction, it may be a simpler matter adequately to drain the underlying foundation material and thus relieve any uplift pressure.

As regards sliding, also, the speaker feels that it is unwise, and, in some instances, has been fatal, to consider a dam apart from its underlying foundation material. The foundation material, or substructure, must act as a unit with the dam body, or superstructure, if the desired object of constructing a safe dam is to be attained. The speaker does not intimate that either of the multiple-arch dams described by the author is in the least unsafe as regards either uplift pressure or sliding. However, he does feel, for instance, that it is not sufficient to aim for a safe shearing resistance at, or near, the base of a buttress, but believes that it is fully as important, and in many cases even more difficult, to design so as to obtain safe resistance to shear or to sliding within the foundation material itself at planes or laminations just below the base of the footings.

That these considerations are not far-fetched is shown by the failures of such dams as those at Austin, Tex.; Lock 26, Ohio River, and Austin, Pa. In the first two, at least, sliding took place within, and not on, the foundation rock.

The speaker has not found in the paper any specific description of the foundation material, but the author characterizes it as "bed-rock worn clean by glacial action", and this statement in itself leads one to believe that the foundation conditions are fortunate, and that probably no harmful laminations exist.

Finally, the speaker believes it would be interesting to others, as well as to himself, if the author were to share with his fellow engineers, somewhat more specifically and in more detail, his views as to the effect of the time interval (elapsing between the occurrence of maximum and minimum temperatures) as regards lessening the formation of temperature cracks.

Mr. Wegmann.

EDWARD WEGMANN,* M. AM. SOC. C. E.—Although the multiple-arch dam has only lately come into prominence, this type was adopted for the Meer Alum Dam, in India, more than a century ago. This dam,

* New York City.

which is about ½ mile long, is built in plan on a curve, convex up stream, and consists of twenty-one smaller arches, having spans of from 70 to 147 ft. The dam is 39 ft. high and has vertical faces on both sides, except near its crest, where the down-stream face is stepped up stream, in order to reduce the top width. Mr. Wegmann.

The Belubula Dam, in New South Wales, was built about 1898. It is 431 ft. long, and has a maximum height of about 60 ft. The lower part consists of a mass of concrete from 1 to 23 ft. deep. The upper part is of brickwork, and has a height of 36 ft. 9 in. In the central part of this brickwork there are six buttresses, 28 ft. from center to center. Between these buttresses there are five vertical, elliptical arches, inclined down stream at an angle of 60 degrees. The spandrels between the arches are filled with concrete which covers the crown of the arches to a depth of 12 in., and joins the side-walls of the dam, which are of concrete. As far as the speaker knows, these two multiple-arch dams are the only ones of this type which were built until recent years.

In 1897, Henry Goldmark, M. Am. Soc. C. E., proposed to build a multiple-arch dam of concrete at Ogden, Utah.* This dam was to be 369 ft. long and 105 ft. high above the foundation at the center of the valley. According to Mr. Goldmark's plan, the dam was to consist of a number of piers, 16 ft. wide and 32 ft. apart in the clear. Alternate bids obtained for building the dam in the manner described, or, as an ordinary "gravity dam", showed a saving of from 12 to 15% in favor of the former plan. Mr. Goldmark proposed to make the thin vertical arches of the multiple-arch dam water-tight by placing a lining of sheet steel on their up-stream side.

About 1900, the speaker was retained to design a multiple-arch dam, 160 ft. high, which was to be built in Virginia, near Washington, D. C. According to the preliminary estimates, this type was found to be about 15% cheaper than a gravity section. Water-tightness was to be insured by coating the up-stream side of the dam with asphalt. Owing to the practical difficulties which might have been involved in building a multiple-arch dam 160 ft. high—a type which, at that time, had only been used for two low dams—the speaker advised his clients to construct a gravity dam. As the promoters of the project could not secure the required financial means, the dam was not built at that time.

George L. Dillman, M. Am. Soc. C. E., has demonstrated, mathematically, the saving that can be effected by building a dam of the multiple-arch type.† He proposed to give the arches between the piers a parabolic section, so as to avoid all re-entrant angles.

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* *Transactions,* Am. Soc. C. E., Vol. XXXVIII, pp. 291, 302 (December, 1897).

† *Transactions,* Am. Soc. C. E., Vol. XLIX, p. 94 (December, 1902).

Mr. Wegmann. It is only of late years that a number of multiple-arch dams have been built.

In 1908 John S. Eastwood, M. Am. Soc. C. E., of Fresno, Cal., built the Hume Lake Dam, in the Sierra Nevada Mountains, as a multiple-arch dam of concrete. The dam is 677 ft. long and about 60 ft. high and consists of twelve vertical circular arches which are supported by thirteen buttresses. At each end of the dam, a wall, like a core-wall, is built into the side of the valley. In 1910 and 1911, Mr. Eastwood built a similar dam about 200 ft. down stream from the Bear Valley Dam, in California. It has a length of 350 ft. and a maximum height of 91.5 ft. above the foundation. In this case, bids were obtained for building a rock-fill dam having a reinforced-concrete curtain, and, also, for an arched gravity dam of concrete. The multiple-arch dam proposed by Mr. Eastwood was found to be the cheapest and was adopted.

A few more multiple-arch dams have been built. Two of these—the Gem Lake Dam, 700 ft. long with a maximum height of 112 ft. above the foundation, and the Agnew Lake Dam, 30 ft. high and 280 ft. long, were designed by the author. Mr. Jorgensen, however, has not only given an interesting account of how these dams were constructed, but, also, a very complete analysis of the stresses in multiple-arch dams. The speaker compliments him on his excellent paper which will be of value to all engineers who have to design such dams.

Mr. Douglas. WALTER J. DOUGLAS,* M. AM. SOC. C. E.—About 7 years ago the speaker's firm designed and built, in the Adirondacks, the Garoga Dam, a multiple-arch structure having a 200-ft. spillway of the gravity type. The concrete arches are 60 ft. high, and their thickness at the top is 2 ft. increasing to 4 ft. at the bottom. There was no difficulty in taking care of the thrust, the foundation being of good slate, strong enough to take the concentrated load transmitted by the buttresses. The economy of the multiple arch was found to be small when its cost was compared with that of the gravity section used for the spillway. The speaker thinks that in most cases where the cost of skilled labor is high and material is cheap, the ordinary gravity type is about as cheap as the multiple arch. The difficulties of constructing thin arches, such as those shown in the paper, often more than offset the saving in material.

The speaker's firm has made a great many designs of dams since the Garoga Dam was built, but has only constructed one other of the multiple-arch type. This was the Peck's Lake Dam, built near the same locality as the Garoga Dam. The height of the arches, above the surface of the ground, varies from 25 to 45 ft. As far as the

* New York City.

speaker knows, this is the only multiple-arch dam which has not been built on a rock foundation. The foundations were of slightly indurated sand and gravel, but not hardpan. The pressure is distributed by reinforced concrete slabs at the toe of the buttresses. As a security against horizontal movements, the arches penetrate about 10 ft. into the ground, where they are loaded uniformly by the reaction of the soil, and, therefore, they are catenary in tension. This part of the arches is heavily reinforced with steel rods, which are well bonded into the buttresses. This arrangement worked out very satisfactorily. Mr. Douglas.

The dams described by the author are very interesting, but, the speaker believes, they are not conservatively designed. All engineers who are interested in the development of reinforced concrete, as applied to structures other than buildings and bridges, know that every once in a while something "gets by." Tanks have been built in recent years, which are 4 in. thick at the top, 8 in. at the bottom, and more than 100 ft. high, but the speaker thinks that is no justification for building similar ones. He believes that both the arches and the buttresses of the dams shown in the paper are unnecessarily thin.

The speaker cannot see how the triangular girder, mentioned on page 417,* will take up the unbalanced thrust of the arches if one of them should fail, and would ask for more detailed information on this point.

The most serious objection which can be raised against multiple-arch dams is the fact that failure of one unit will destroy the stability of the whole structure. The speaker believes that an occasional abutment buttress, similar in purpose to the abutment piers of arch bridges, consisting of a series of spans should be introduced in the design of multiple-arch dams having a large number of arches.

On page 416,* the author says:

"The total vertical load on the section shown in Fig. 6 is seen to be 6 332 tons, and the horizontal water pressure, 5 062 tons, both on a 40-ft. span. If the coefficient of friction is taken at 0.75, it is seen that the actual shear along the base amounts to only $5\,062 - 6\,332 \times 0.75 = 313$ tons. There is considerable steel in the section to help take up this shear, and, therefore, it was not deemed necessary to eliminate the shear entirely."

This statement, the speaker believes, is incorrect. The shearing resistance of an elastic solid is an elastic attribute of the material. The deformation caused by shearing forces, if within proper limits, will disappear after the actuating forces have been discontinued. The frictional resistance is of an entirely different nature, and can act only after the section has failed under shear. The resisting friction can be developed only by minute, but measurable, movements of adjacent planes. This movement is progressive, that is, the dislocation

* *Proceedings*, Am. Soc. C. E., March, 1917.

Mr. Douglas. caused by the forces resisted by friction will not disappear when the structure is relieved of load, and a second application will necessitate a further movement. In designing one should rely on shear or on friction, and not on both.

As regards the determination of the direct stresses, the speaker would state that the stress computed by the author on page 417,* acting on planes at right angles to the resultant, is not necessarily the maximum compressive stress, and that methods are now available for computing the principal stresses in a buttress quite accurately.†

The speaker has made the foregoing criticism because, as the author correctly states on page 402,* "the stresses and dimensions can be calculated with great accuracy", in multiple-arch dams, although such calculations are almost impossible in earth or rock-fill dams. The determination of the stresses is subject to many uncertainties, even in gravity dams. The design of this latter type, however, is being developed along scientific lines with very satisfactory progress, and the gravity dam has the advantage of a great number of precedents over the multiple-arch dam.

In the case of solid impervious rock foundation, the speaker decidedly prefers the gravity dam, and doubts whether material saving, if any, can be obtained by the use of the multiple arch, even if this latter would have a considerably smaller volume. The unit price of masonry for the multiple-arch dam is out of all proportion to that of the gravity dam, as illustrated by the price of $22 per cu. yd., given by the author.

If the foundation is fissured rock or other good but pervious material, the multiple-arch dam, generally, should be the more economical type, because the volume of the gravity dam must be increased, in order to meet the uplifting action of the water.

Mr. Duryea. EDWIN DURYEA,‡ M. AM. SOC. C. E.—The speaker's home and most of his professional practice are in California, and he is familiar with the general climatic and hydrographic conditions throughout the State and in the Sierra Nevada Mountains, though not with those of Mono County, in which are built the two dams forming the subject of this paper. However, hydrographic conditions are known to be quite uniform throughout the high Sierras of California, and presumably in Mono County they do not differ materially from those of the parts with which the speaker is familiar.

A previous speaker has expressed doubts as to the adequacy of the spillway arrangements of these two dams to pass safely such intense floods as might be expected to occur at long intervals from such a small drainage area as 22½ sq. miles. In most parts of the United States,

* *Proceedings*, Am. Soc. C. E., March, 1917.

† See the papers of Levy, Unwin, Hill, Ottley, and Brightmore.

‡ San Francisco, Cal.

such apprehensions would be justified, but not in the high Sierras of California. Mr. Duryea.

Throughout that State practically all the year's precipitation occurs during the winter, from about October to about March, inclusive, with hardly any (and none such as will cause floods) during the warmer months of April to September. Hence, in the high Sierras, the year's precipitation is practically all in the form of snow, which accumulates throughout the winter to depths of even 20 ft., or more, settles and packs until melted gradually by the increasing temperatures of the spring and summer, and passes off as stream flow in the form of slowly rising floods, which usually begin in April, culminate in June, and terminate about the middle of July. It is this condition of winter rains in the valleys, which soak the ground and start the growth of the crops, in conjunction with the snow stored in the high Sierras, which by its gradual melting furnishes water for irrigation throughout the growing season of April to September, that causes the very great success of irrigated agriculture in the Great Valley of California.

On the drainage area above these two dams (at elevations of from 9 000 to 12 000 ft. above sea level), and even at much lower elevations, this condition of stream flow from the slow melting of the snow must be marked and invariable; and on that area there can be no direct connection between excessive storm intensities and excessive flood intensities.

The speaker has examined many dams in the high Sierras of California and, at first, was strongly impressed by the apparently very inadequate provisions for spillway capacity and "free-board" of those dams, as judged by necessities elsewhere. Soon, however, he realized that provisions which would have been very inadequate and unsafe in most other regions, are quite safe in the high Sierras.

A previous speaker has mentioned the small number of arch dams as yet built in the world, placing the number at only about 15. The speaker has built an arch dam which may not be included in that number. It is the Goodwin Dam, on the Stanislaus River, not far from Stockton, Cal., and serves to raise and divert the waters of that river for the irrigation of the South San Joaquin and Oakdale Irrigation Districts, which together comprise about 144 000 acres. The dam consists of two arches with a buttress between them; there is also a small buttress at the south end of the dam to replace a rock abutment which was badly shattered by the blasting of a canal. The south arch is 299.4 ft. in crest length, with a maximum height of 78 ft.; the north arch is 159.8 ft. in crest length, with a maximum height of 42 ft. The buttress between them is on a ridge of rock (slate) near the middle of the river, and has a maximum height of masonry of only 22 ft.

The two arches have radii (of their vertical up-stream sides) of 135 ft., and are proportioned for "pure arch action" by the ordinary

Mr. Duryea. arch formula and for maximum arch stresses of 300 lb. per sq. in. (43 200 lb., or 21.6 tons per sq. ft.). Except that the minimum arch thickness (in the upper parts of the arches) was fixed arbitrarily at 8 ft., the arches were thickened 4 ft. arbitrarily at the rock surface (this thickening decreasing to zero 8 ft. above), as a safeguard against possible erosion by the very turbulent waters of the pool just below the dam. The crest was made of a somewhat unusual shape to permit larger flows over it without reducing its strength, and the arches were reinforced against temperature stresses by steel bars, near their crests and for 6 ft. below, near their up-stream and down-stream sides. The thickness of the south arch, 59.8 ft. below its crest (just above the thickened base), is 12.2 ft.; that of the north arch, at the corresponding point, 24.5 ft. below its crest, is 8.0 ft.

The arches are proportioned (aside from the exceptions noted) for 300 lb. per sq. in. and for the maximum arch thrusts caused by floods up to 25 ft. in depth over the crest, making allowance, however, for the relief of the arch pressure by the variable "back-water" against the down-stream sides of the arches. Two measured floods are known (in 1907 and 1911) which would have caused depths on the crest of the dam (had it then been built) of about 15 ft. The dam was completed in 1912, and since then several floods of about 6 ft. in depth have passed over it. The drainage area above the Goodwin Dam comprises about 928 sq. miles of mixed low and high Sierras, from an elevation of 350 ft. above sea level (the crest of the dam) to an extreme height of more than 10 000 ft.

The first requirement in any dam should be certain safety, but the next should be economy; and the usual reason for designing an "arch" dam is that thus considerable saving below the cost of a "gravity" dam often may be effected, without (at least in the judgment of the designer) any sacrifice in safety. The Goodwin Dam is a good example of such a saving in cost. Six other dam sites were surveyed, three up stream and three down stream from the Goodwin site. Estimates of cost were made for "gravity" dams on all seven sites, and for three other plans of arch dams on the Goodwin site. By the choice of the Goodwin site and the two-span arch dam adopted and built, the saving effected was more than half that of the most obvious procedure—the raising of an existing gravity dam 1¾ miles up stream, already the property of the two Irrigation Districts, the enlarging of the intervening 1¾ miles of their existing canal on the north side of the river, and the building of the same length of new canal on the south side.

In 1903, the speaker designed two other arch dams for California streams, neither of which, however, has been built.* The larger one was to have been 138 ft. in maximum height, and made up of eight semicircular arches, supported on intervening buttresses 50 ft. from

* *Transactions,* Am. Soc. C. E., Vol. LIII (December, 1904), pp. 172-175.

center to center. The backs or up-stream sides of the arches were to have been vertical, with radii of 26.5 ft. The smaller dam was to have been 30 ft. in maximum height, with eight spans of $19\frac{1}{2}$ ft. from center to center of buttresses. The "deck" or up-stream side of the dam was to have been a plane with an inclination of 0.64 vertical to 1.00 horizontal, in order to distribute the foundation pressures more evenly over a rather weak rock foundation. The crown thickness of the deck was to have been 2 ft., and the radii of the inclined semicircular arches forming the under side of the deck were to have been $8\frac{1}{4}$ ft. Mr. Duryea.

Mention is made in the paper, and by a previous speaker, of the freedom of multiple-arch dams (or other thin dams) from "uplift" forces. Such forces are quite certain to exist in greater or less degree under all thick masonry dams. The speaker recently had an opportunity to measure the uplift forces existing under parts of the base of a high masonry dam (the La Boquilla Dam, in the State of Chihuahua, Mexico, of 244 ft. maximum height). The measurements extended over a period of 22 months, and for depths of water behind the dam (above the foundation level) of from 140.3 to 213.6 ft.; and comprised measurements in thirty-nine holes, aggregating sixty-three or more hole measurements of the uplift. He hopes soon to present these measurements in full to the Society; and will only say now that, although the foundation of that dam, in its natural condition, was a very good limestone, with but few and minor defects (which were all repaired with great care and cost by cement pressure grouting), still the measurements of the pressures show the existence of "uplifts" which, in general, lie on straight lines decreasing from (at the "heel" of the dam) one-third of the depth of water in the reservoir above the foundation plane, to zero at the "toe" of the dam. Incidentally, the dam had been designed to withstand uplifts varying from two-thirds of the depth of the water at the heel to zero at the toe.

Regarding the use and safety of arch dams, one general thought is worthy of serious consideration: Several (only a small proportion, however) "gravity" masonry dams have failed. It is true that these failures were probably always from faults perceived afterward to be due to recognized poor design or construction, and that they would not have occurred had the design and the construction of the profiles and the foundations been in accordance with recognized good practice.

On the other hand (even though most arch dams are much thinner than gravity dams, and though there should be no reason to suppose their profiles and foundations to have been in general designed and constructed more carefully), the speaker does not recall, among the few arch dams as yet constructed, even a single failure.

It is realized that such reasoning is not conclusive, but, nevertheless, it appears to the speaker at least probable that the "arch" dam possesses greater reserves of strength and safety than the "gravity" dam.

Mr. Nishkian.

L. H. Nishkian,* Assoc. M. Am. Soc. C. E. (by letter).†—The author has rendered to the Engineering Profession a much needed service by opening up for discussion the subject of multiple-arch dams.

There is no question that, in the future, dams of this type will prevail over other types in many locations, and, therefore, a detailed discussion of methods of design and construction is timely and profitable.

At first glance the structural design of a multiple-arch dam seems to be quite simple, requiring the design of a concrete arch under definite forces, of a buttress to uphold the arch, and of a foundation to hold the buttress in place.

On closer study, many problems arise. The upper and lower ends of the arch barrel are difficult of even roughly approximate analysis, and the designer has to depend largely on his judgment. The weight of the arch between the buttresses is carried partly on the latter and partly on the lower end of the arch. In designing the arch, the author has considered its weight component normal to the buttresses as being carried by arch action to the buttresses, and, presumably, the component parallel to the buttresses as being carried by direct column action down to the footings at the lower end of the arch barrel. In the design of the buttress and foundation, however, it has been assumed that the entire weight of the arch is acting, through its center of gravity, on the buttress.

The writer agrees with Mr. Jorgensen in regard to the method of designing the arch ring, but cannot agree with him in throwing the entire weight of the arch on the buttresses. It appears to the writer that only the weight component normal to the buttress should be considered as acting on it. These two assumptions are widely at variance, and result in a large difference, both in the point of application and in the line of action of the resultant force on the buttress.

The author finds an upper and a lower limit for the direct compression stresses in the buttress. It will be found, however, that the stress produced by the normal component of the arch thrust on the buttress is equal to, or greater than, the maximum stress given in the paper. In this case, the arch thrust, due to water pressure at a depth of 80 ft., is approximately $80 \times 62.5 \times 23.08 = 115\,000$ lb., and the thrust, due to dead weight of arch ring, is approximately $150 \times 3.7 \times \cos. 50° \times 23.1 = 8\,000$ lb., or a total arch thrust of 123 000 lb. This will result in a force of $123\,000 \times 2 \times \sin. \frac{119° 57'}{2} = 213\,000$ lb. per lin. ft. on the buttress. The buttress is here 4.25 ft. thick; therefore, we have a stress of $\frac{213\,000}{4.25 \times 12 \times 12} = 348$

* San Francisco, Cal.

† Received by the Secretary, May 18th, 1917.

lb. per sq. in. Although in no sense dangerous, this is in excess of the stress of 317 lb. per sq. in., given by the author as the maximum existing in the buttress. One must be careful in using average values of stresses where the average is for a large area. Mr. Nishkian.

The author has made the section of the arch normal to the up-stream edge of the buttress, circular. As is pointed out in the paper, the pressures are not uniform on the arch ring, with the result that, near the top, it is necessary to move the arch out 5 in. at certain points, in order to make its center line coincide with the line of pressure. To avoid this difficulty and maintain a constant shape from top to bottom, it is only necessary to make a horizontal section of the arch circular in shape. A normal section will now be an ellipse, and it will be found that the center line of the arch ring and the line of pressure will coincide very closely for all depths.

It will be necessary to use a three or five-centered curve for the form ribs, and this will entail an extra expense, as the circular form ribs will have to be made in sections also. Furthermore, with a horizontal section circular, the up-stream and down-stream edges will be concentric, with a uniform radial thickness of arch ring. This will result in the normal section of the arch ring being much thicker at the buttresses than at the center, giving it greater stability.

The author has provided a light but very effective method of stiffening the buttress walls, and it is worthy of more general use.

Gardner S. Williams,* M. Am. Soc. C. E. (by letter).†—No discussion of the multiple-arch dam would be complete without mention of the famous Meer Alum Dam,‡ near Hyderabad, India, built about 1800, which is made up of a series of full-centered vertical brick arches with radii varying from 70 to 147 ft. These arches are arranged in the arc of a circle, having a crest length of about ½ mile and a height of about 40 ft. This structure appears to be the dean of multiple-arch dams. Mr. Williams.

On September 1st, 1902, the writer tabulated the bids received by the Ithaca Water Works Company, and a few days later awarded the contract for the construction of the so-called Six-Mile Creek Dam, near Ithaca, N. Y. This dam embodied in its design a section of the inclined multiple-arch type and is believed to be the first structure of the kind for which a contract was ever let.||

Prior to the design of this dam, which was made by the writer in the early part of 1902, plans and specifications for an inclined multiple-

* Ann Arbor, Mich.

† Received by the Secretary, May 26th, 1917.

‡ *Transactions*, International Engineering Congress, 1915, Waterways and Irrigation, p. 719.

|| The Six-Mile Creek Dam was discussed in *Transactions*, Am. Soc. C. E., Vol. LIII, and the multiple-arch section is particularly described on page 195 thereof.

Mr. Williams. arch dam were prepared by Henry Goldmark, M. Am. Soc. C. E., for the Pioneer Power Company, of Ogden, Utah,* the suggestion of this design being due, the writer is informed, to George H. Pegram, President, Am. Soc. C. E. The inclination of the arches was about 25° from the vertical. Subsequently to the work of Mr. Goldmark and prior to 1902, the multiple-arch dam was investigated by James H. Fuertes, M. Am. Soc. C. E., and by the late Emil Kuichling, M. Am. Soc. C. E., independently, but neither of their designs came to the constructive stage.

Since the construction of the Six-Mile Creek Dam, dams of the multiple-arch type have been designed and built under the supervision of the writer in Michigan, Minnesota, and Iowa.

The principles of design laid down by the author differ somewhat from those followed by the writer, and appear to transgress the rules of economical design, since the structure, as described by the author, does not follow the equilibrium curve. As is well known the curve of equilibrium for an arch under uniform normal pressure is a circle, and, therefore, since the inclined arch may be conceived as made up of an infinite number of infinitesimally thick horizontal arches, the horizontal section of the arch should be a circle and not an ellipse, as seems to have been contemplated by the author. A section of the arch normal to the axis will then be elliptical, and the inclination should be chosen so that this ellipse approaches most closely to the hydrostatic arch for the condition of most constant load. When designed in accordance with these principles, the arch is subjected to the minimum of bending stress, and requires the least material. An increasing thickness of the arch section to the bottom of the dam is hardly necessary, as over the lower one-third, in ordinary cases, the beam action is such as to relieve to a considerable degree the arch action, so that the shell may very properly be made of uniform or even diminishing thickness from the point of one-third the height down.

None of the dams built by the writer has been on a rock foundation, and although it is necessary to be assured of the carrying capacity of the material under the buttresses, this does not preclude the use of such structures on sand, gravel, or clay.

Mr. Howson. GEORGE W. HOWSON,† ASSOC. M. AM. SOC. C. E. (by letter).‡—Mr. Jorgensen has presented a very excellent paper on the design and construction of multiple-arch dams. The method of working out the design shows much careful study, and appears to be the last word in the design of dams of this type.

There are several phases of the paper, however, which hardly seem to be in accord with the writer's experience, and there is a general

* *Transactions,* Am. Soc. C. E., Vol. XXXVIII, pp. 290 to 297.

† San Francisco, Cal.

‡ Received by the Secretary, August 1st, 1917.

tendency to compare the multiple-arch with the rock-fill dam. These are two entirely different types, each having its own advantages. Mr. Howson.

In general, it may be stated that a rock-fill dam may be built on any foundation suitable for a multiple-arch structure; but the reverse is not true. As an example, a rock-fill dam is best adapted to a site where the underlying bed-rock across the floor of the canyon is covered by, say, from 20 to 30 ft. of over-burden. Here it may be necessary to carry only the thin cut-off wall down through the overlying material into solid rock; and the great bulk of the dam, the loose drop fill, may be built on the over-burden. It must be known, of course, that the foundation under the rock fill is suitable, though it may not be a foundation on which the buttresses of a multiple-arch dam should be built. Mr. Flinn has brought out this point in comparison with a solid masonry dam, but his comparison of economy relative to the foundation is even more applicable in the case of a rock-fill structure.

It is then seen that the problem of choice between rock-fill and multiple-arch types presents itself only at a site where a good foundation for the buttresses is exposed, or is found within a limited depth of excavation. Here the final economic balance may be in favor of either type, depending on the particular cross-section of the canyon, the transportation problem, and the general topography of the site. The writer, however, believes that, in a majority of cases where it is possible to consider a choice of types, the relative cost of a multiple-arch dam and a rock-fill dam will be in favor of the former.

Mr. Jorgensen is correct when he states that the water-tightness of a dam depends to a much larger extent on the quality of the concrete in the arches than on the thickness of the latter. The Strawberry Rock Fill Dam, built recently by the Sierra and San Francisco Power Company, may be said to be bottle-tight, although it depends on a reinforced concrete apron only 18 in. thick, under a 130-ft. head, which thickness tapers to 9 in. at the crest. A multiple-arch dam should be built only under the most competent engineering supervision. A few failures of dams of this class due to poor construction would condemn all such structures in the public opinion.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

AN AERIAL TRAMWAY FOR THE SALINE VALLEY SALT COMPANY, INYO COUNTY, CALIFORNIA

Discussion.*

By Messrs. Richard Lamb and H. F. Scholtz.

Richard Lamb,† M. Am. Soc. C. E.—On studying this paper, with reference to other plants which have been built, it is seen that it describes novel features and conditions which were overcome with difficulty. Mr. Lamb.

At first one might wonder that such an expensive plant should be constructed for handling so common and cheap a commodity as salt, for a 7½-lb. bag of the best grade retails at present for 10 cents, and lasts a family for quite a time. During normal times much salt is brought to the United States as ballast for vessels, and is sold at a low price.

Recently, the speaker was examining a cement stone deposit in Alabama, and found, at a point 3 miles from tide-water, a salt deposit from which most of the salt used by the Confederate States was mined during the Civil War. A comparison of the plant described by Mr. Carstarphen with a project to mine salt and deliver it by cableway to tide-water from the Alabama deposit indicates that the latter would have a great advantage. Later, the speaker visited the Gulf Coast of Louisiana, and there found that vast deposits of salt are now being mined and shipped by boat or train with little or no intermediate transportation, and the owners cannot sell all they can mine. However, it is presumed that the market was examined before the Saline Valley cable tramway was built.

* This discussion (of the paper by F. C. Carstarphen, M. Am. Soc. C. E., published in April, 1917, *Proceedings*, and presented at the meeting of May 2d, 1917), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† New York City.

Mr. Lamb. One of the novel features of this tramway is its excessive gradients; the speaker knows of no other long cable tramway where they are so steep. The only other plant in the United States which can compare with it in length is that of the American Copper Company, in Wyoming, 16½ miles long. In the latter, however, the maximum vertical angle is about 20°, as compared with 40° in the Saline Valley plant.

The undertaking was bold, in that such gradients had not heretofore been overcome, and the success of the plant would depend on the efficient service of the grip used. The speaker would have thought that the Pohlig grip would have been successful; however, the author states that the plant was not equipped with either the Pohlig or the Bleichert grip.

In the Bleichert system the difficulty, on steep gradients, is that in going up the steepest incline of the cable, just before surmounting the saddle at the tower, the hauling cable is pulling somewhat downward, and the resultant force has a tendency to retard the carriage. In the system designed by the speaker, the hauling cable is kept parallel with the bearing cable, and the downward pull is practically obviated. This feature is used on the line of the American Copper Company.

The speaker's discovery of this principle was due to the following incident. A patented logging cable tramway had been erected by parties who stated that it was practical. The bearing cable was carried on A-frame supports. A trolley carriage ran on the cable, and from it the log to be transported was suspended. Attached to the log was a cable which was made fast to a whippletree drawn by a horse. As the trolley approached the cable support, it had to ascend the incline caused by the inherent sag in the cable. This direction was one line of force in the parallelogram of forces. The direction of the pull by the horse was horizontal. The resultant force, consequently, was downward, and the trolley would not pass over the support. The project was a failure, and the difficulty could not be overcome under the conditions of the system.

If a trolley trackway is on a grade and a rope is attached to the trolley to pull it in a horizontal direction up grade, the trolley will not move. It is necessary to have the hauling and bearing cables as nearly parallel as possible. The steeper the gradient, the more difficult this problem becomes. Practical plants embodying this principle were described by the speaker some years ago in a paper* before this Society. An inherent feature of cable tramways is that they must be built in an absolutely straight line. In the logging cable tramway described by the speaker, in the paper referred to, the sheaves carrying the hauling cable were hung so as to permit the

* *Transactions*, Am. Soc. C. E., Vol. XXXII, p. 44.

alignment to deviate from a straight line, but it had to be very nearly straight. Mr. Lamb.

In the Saline Valley line there are three changes of direction, or four separate cable tramways joined by curved rail at the points of deflection, with electric motors to operate the hauling cable at each angle. At each control station the traction cable is made a unit for that station only, and is driven by its own motor. The summit station is arranged so that both the grip sheaves are on the same shaft, and are controlled by a motor which supplies the difference between the power required to hoist the ascending bucket and that developed by the descending bucket. The power is not transmitted around curves, as in surface cable-power street railroads.

The tension ordinarily used on cable tramways is about 30 000 lb. per sq. in. of cable. The cable can never be stretched to approach closely the horizontal; if so, it would be strained beyond its elastic limit. The towers are usually about 250 ft. apart. Tension stations are about 5 000 ft. apart. In some cases spans as long as 1 600 ft. between towers have been operated successfully.

The Chilcoot Pass cableway was built mainly for the transportation of passengers. For years this Pass had been traversed by miners and others seeking gold, and it was so difficult that many lives were lost there. The Trenton Iron Company built a cableway that carried freight and passengers successfuly over the Pass. It was intended to build the power plant on a plateau, named White Horse, near the center of the Pass, but an avalanche buried the site, and it was with difficulty that another could be found.

The power was furnished by a porcupine boiler in which the tubes were of such weights as could be carried to the site by Indians. The bearing cable could not be more than $\frac{5}{8}$ in. in diameter, as it also had to be transported by Indians. The transportation from Trenton to the final location was more expensive than the material of the plant. Great care had to be exercised in making the cable, as there was one span about 1 500 ft. long. The first person to traverse the line was a girl who used her canoe as a car.

H. F. Scholtz,* Assoc. M. Am. Soc. C. E.—There are several places in the Pacific Northwest where tramways for transporting logs are in successful operation. The logs are taken out from places practically inaccessible by any other method. One instance is in the Cascade Mountains, near Index, Wash., where logs as large as 6 ft. in diameter are carried across a deep ravine and down a precipitous cliff to the mill. Mr. Scholtz.

* Carrollville, Wis.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

This Society is not responsible for any statement made or opinion expressed in its publications.

MODERN PRACTICE IN WOOD STAVE PIPE DESIGN AND SUGGESTIONS FOR STANDARD SPECIFICATIONS

Discussion.*

By Messrs. Hermann von Schrenk, Frank F. Bell, D. C. Henny, Henry P. Rust, F. M. Robbins, William J. Boucher, and J. C. Ralston.

Hermann von Schrenk,† Assoc. Am. Soc. C. E. (by letter).‡— This paper has been read with much interest by the writer, but he finds that it contains certain rather startling errors relating to cypress, its method of growth, and its grade. Mr. von Schrenk.

On page 566§ the author states: "A cypress tree is the product of four or five small trees growing together." Furthermore, he explains that, as a result of such growth, the sapwood does not occur around the circumference of the tree, but in streaks throughout the trunk. This is so startling an error that it should not go unchallenged. The cypress does not grow as stated by the author, but like all other trees. It starts from a seed, and develops a trunk as it grows older; and the sap ring, under all circumstances and conditions, is on the outside, immediately under the bark, just as in practically all other trees. Having studied the growth of these trees for about 25 years, under every conceivable condition, the writer has yet to find a single instance of any such occurrence as is mentioned in the paper. Individual trees formed by the union of several sprouts are possible, of course, but these would be more likely to occur in redwood than in cypress, because, unlike the redwood, the cypress does not sprout from the stump. The

* This discussion (of the paper by J. F. Partridge, Jun. Am. Soc. C. E., published in April, 1917, *Proceedings*, and presented at the meeting of May 16th, 1917), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† St. Louis, Mo.

‡ Received by the Secretary, May 7th, 1917.

§ *Proceedings*, Am. Soc. C. E., April, 1917.

Mr. von Schrenk. possible reason for Mr. Partridge's mistake is indicated in his discussion as to sap streaks in the wood. He states: "It is common to see clear cypress with yellow sap streaks running through the center at intervals of about 4 in." From this sentence, it would appear that the author interprets the lighter streaks of color in the heartwood as sap streaks. The color of the cypress heartwood varies from very pale yellow to almost black, and this may occur in one tree or in different trees growing in the same region. This variation in color has given rise to all sorts of common names, such as white cypress, yellow cypress, black cypress, etc., and it is very common to find these curious color differences or markings in the same trunk. These color variations, however, have nothing to do with the sapwood. There is never any question about identifying the sap ring, which in cypress is usually very narrow, particularly in the older trees. The curious color variations or streaks in the cypress have been ascribed to the variation in absorption of material from the soil. The writer has frequently noted that in certain regions one type of color variation will appear and in other regions another type. The writer would be much interested to have a case pointed out where a trunk actually is composed of more than one original seedling, because this certainly would be a scientific curiosity. If Mr. Partridge's statements were true, what would become of the bark on the inside, if several trees fused?

Attention is also called to a slight error in Mr. Partridge's description of grades. Although it is true that, in the listing of the grades by the Southern Cypress Manufacturers' Association, the grade "Tank" appears first in the book of specifications, as a matter of fact Grade "A" is really higher than "Tank." In cypress there is no such grade known as "Clear." The manufacturers of cypress regard the Grade "A", in the heavier thicknesses, as being better than the "Tank" grade in the same thickness, for the reason that knots are eliminated on the face side of "A", whereas any number of water-tight knots are admitted in the grade of "Tank."

Mr. Partridge is probably correct in his surmise that 4-in. "Tank" stock is better material than 4-in. Grade "A" for pipe staves, for two reasons: First, an occasional piece of Grade "A" would have a very slight quantity of sap on the reverse side, whereas "Tank" would be strictly free from sap, although it would contain knots. Second, the "Tank" stock, in the heavier thicknesses, can be purchased at a lower price than the Grade "A" of the same thicknesses. As indicated by Mr. Partridge, the mere presence of knots does not influence in any way the value of "Tank" stock for stave purposes.

Attention is also called to a slight variation in the author's nomenclature from that generally accepted as standard by such organizations as the American Society for Testing Materials, the American Railway Engineering Association, and the Forest Service and other Govern-

ment bureaus. The terms usually applied as standard for the two grades of wood formed by a tree during the year are "springwood", for the soft wood formed early in the year, and "summerwood", for the hard wood formed later in the year. Many readers will doubtless confuse Mr. Partridge's names of "summerwood" and "winterwood" for the more usual terms of "springwood" and "summerwood." Mr. von Schrenk

The writer would like to ask the author for a further explanation as to why penetration is insisted on for a stave. The writer may not understand the meaning of the word "penetration", but assumes that it means the thorough absorption by the stave of water or whatever liquid is passed through the pipe. If any figures are available, as the result of tests, it would be of interest to know on what basis the statement is made that a dense piece of pine, for instance, will not make as good a pipe as a piece of redwood, or other material, of similar density.

Frank F. Bell,* Esq. (by letter).†—The following general ideas along the lines of this paper are based on the writer's experience in wood stave pipe design and construction. Mr. Bell.

The writer finds it to be generally true, as Mr. Partridge has outlined, that there is little authentic knowledge among engineers relative to either continuous-stave or machine-banded pipe, as it is considered somewhat of a specialty, although coming more and more into use in modern engineering practice.

Ideas regarding such pipe are at variance in different parts of the country. In general, the specifications and design are under the influence of the locality in which the engineer intends to build. In the Northwest, we find strong adherents to pipe made from Oregon pine and Douglas fir; in the Southwest, redwood; and in the East, Canadian pine and yellow pine. This is due mostly to the sales talk of the representatives of the various manufacturers of pipe, and the engineer has to be guided more or less by what he hears rather than by actual experience or standards. Wood stave pipe has passed the stage where it can be considered as a specialty; and it is certainly time that general practice and standard specifications should be worked out.

The author's presentation of the subject is complete, but, of course, with reference to the specifications, there may be more or less disagreement. As a start, however, the writer believes the paper will furnish a reasonable basis for the best practice.

It is generally admitted that, for purposes of durability, redwood is logically the best. The writer does not believe there will be any discussion on this point. He has heard engineers criticize this lumber on account of its softness, but, for wood pipe, such criticism is hardly worth considering, as the strength depends on the band spacing, and

* President, Simplex Vacuum Mfg. Co., Philadelphia, Pa.

† Received by the Secretary, May 16th, 1917.

Mr. Bell. when the pipe is made up, there seems to be no material difference in rigidity when redwood and harder woods are compared.

The author does not mention yellow pine as a good lumber for wood stave pipe, but this is hardly worthy of important notice, as such wood is too scarce to be a logical material, and would be put in a class with cypress.

In the eastern part of the United States, the general practice is to use thick staves and flat bands for machine-made pipe, but in the West round steel wire is used. A selection between these two styles should be based on local requirements. For continuous-stave pipes, the eastern style is not in keen competition. It is mostly in specifications for pipe of large diameter that standard practice is needed.

The writer agrees with Mr. Partridge that redwood need never be given a coat of preservative, provided clear lumber is used, according to his specifications. On the other hand, in pipe constructed of other kinds of lumber, a preservative coating is necessary, if longevity is wanted. Redwood, under all general conditions, will easily outlast bands and wire winding, without any preservative application.

The writer is familiar with the North Yakima development, mentioned in the paper, as well as the sewer at Palo Alto, Cal. The North Yakima pipe underwent about as rigid a test for durability as would be found in any practice, and, from studying this particular case, it appears that the drying of the lumber is one of the most important factors. The process of kiln-drying *versus* air-drying is, of course, familiar enough to all engineers; but the writer again agrees with Mr. Partridge that, if possible, air-dried lumber should be used, although very good results are obtained with kiln-dried fir.

In the matter of machine-banded pipe, the drying is even a more important factor, because there is no chance to "cinch up" in case of shrinkage, and the strength and durability depend on the bearing surface and winding tension of the wire.

The author's remarks on the importance of butt joints should also be emphasized, as in continuous-stave pipe this is generally where trouble starts first, and it is also one of the drawbacks in having wood pipe built by men who do not understand the finer details of construction.

The writer knows of a case (which happened within the last few months) of a contractor who had his own staves milled by the ordinary process, bought his own bands, and attempted to put in his own pipe according to his idea as to specifications and methods. This pipe, no doubt, will be unserviceable in a very short time, and, consequently, engineers who come in touch with this particular case will be influenced against wood pipe in general. This is why the writer believes that the author's remarks should be given favorable consideration. If standard specifications and practice are put in the hands of engineers, there will

be a much more favorable attitude in general toward wood pipe, and it will be given the place it deserves as an engineering utility. Mr. Bell.

Mr. Partridge's classifications are very fair for all kinds of pipe. Naturally, the redwood man claims that his pipe is the best, the fir man his, the pine man his. The only way for the engineer to decide is to make a comparison of numerous structures. On the basis of data gathered from a number of wood stave pipes, it is thought that Mr. Partridge has rounded out the matter very well. The specification tables in the paper would be what the writer would use if he were designing a pipe line.

The writer has found by experience in many cases that a wood stave pipe has cost the designing engineer much more than it would if he had used the most economical specifications; and, in a majority of cases, the engineer has lost, in the long run, by sacrificing price to permanency, by failing to adopt the specifications and the style of pipe needed for the conditions under which he is designing, having been talked into "something just as good."

If careful unprejudiced consideration is given to this paper engineers will feel that standards should be generally recognized, and that the proper styles, classifications, and lumbers should be given level-headed consideration. As it is, all are depending too much on the influence of representatives who are talking up the various kinds of pipe and, naturally, where one lumber predominates, the designing engineer will be influenced in his specifications.

D. C. Henny,* M. Am. Soc. C. E. (by letter).†—From a report by the writer to the U. S. Reclamation Service in July, 1915, on the life of wood pipe, the author has quoted a tabulation‡ in which an attempt was made to condense the information collected from past experience and special investigation. It is well to quote also from the two paragraphs in the report immediately preceding this tabulation, which read as follows: Mr. Henny.

"The investigation has had in view especially the life of pipe as affected by the durability of the wood. * * * The information collected is based mostly on reports received from managers or owners, and in small part on personal observation. It is not as complete as is desirable, which, however, is not due to failure to elicit further information.

"Reviewing the information as grouped under its headings, it may be estimated that under conditions of continuous water pressure the life of various kinds of pipe may be as follows:"

The last item of the tabulation refers to the estimated life of wood in fir pipe, well-coated, buried in loose soil, and maintained under

* Portland, Ore.

† Received by the Secretary, May 16th, 1917.

‡ *Proceedings*, Am. Soc. C. E., April, 1917, p. 566.

Mr. Henny. continuous water pressure. Some additional information has come to hand since the date of the report, which, although not directly applicable, may be considered of some importance.

During October, 1916, the writer made an examination of wood pipe lines—part of an irrigation system in the vicinity of Pasco, Wash. —constructed in the spring and early summer of 1910. The pipe portion of the system consists of 4½ miles of continuous-stave, uncoated, fir pipe, 30 and 36 in. in diameter, and 12½ miles of machine-banded, coated, fir pipe, from 10 to 24 in. in diameter.

Decay in the uncoated pipe had made it necessary to rebuild 8% of the line during 1915 and 23% during 1916. Examination in October of that year showed that serious decay had then affected the remaining 69% of the pipe. It had attacked principally the bastard grain wood and wood with a coarse vertical grain, such as is derived from the top of the tree. No serious decay was found in the close-grained wood with vertical grain, and staves of this kind had been used in the reconstruction.

This is stated merely as incidental to experience with machine-banded pipe, repairs on which, up to the time of examination, had been confined to the replacement of wood sleeves. Only a small percentage of this pipe was open to inspection. This, however, showed that some of the wood in the pipe itself had been attacked, the decay being confined largely to individual staves in the upper half of the pipe. To what extent the condition of the small portion of pipe examined may be indicative of that of the remainder is problematical.

It was noted that, in several places where decay had occurred, there were in evidence hundreds of insects resembling ants. Whether these were cause or effect could not be determined.

All the pipe is buried in loose, sandy soil. The winter climate is mild, and the summer is hot, the temperature frequently rising above 100 degrees. The coating appeared to be the manufacturers' standard, consisting of asphalt and tar.

This pipe is not included under the conditions predicated in the tabulation quoted, as during 5 months of the non-irrigating season it is not maintained full. It is probable, however, that this fact is only in small part responsible for the decay observed. It will be necessary to examine a larger portion of this pipe, or await the results of experience with it, before dependable conclusions can be drawn. Nevertheless, its early decay in spots affords ground for reducing the life estimate, of well-coated fir pipe under very unfavorable conditions, from the figures given in the tabulation to 10 years or even less.

The author has presented his case with clearness and ability, and the virtues of redwood in pipe construction are well brought out.

Although the paper mentions the variability of wood of the same species, it does not specifically deal with the variations to which red-

wood itself is subject. The wood in the lower 30 ft. of a large redwood tree is exceedingly close-grained. It sinks in water and is remarkable for its long life. Fence posts made of it have lasted more than 50 years. Mr. Henny.

Higher in the tree, the grain gradually becomes coarser, and near the top it shows fewer than ten rings to the inch. It also becomes lighter, and, although no specific tests are known to the writer, it is probable that the life of redwood as well as fir becomes shorter as the grain becomes coarser. In the past, specifications have sometimes called for sinker redwood, but, from a practical mill standpoint, this is almost impossible to supply in large quantities. However, in scientifically drawn specifications, it may be in point to limit the coarseness of the grain, the more so as the superior lightness of the coarse-grained wood offers a temptation for its selection (where freight charges become an important item), to a contractor not concerned about the ultimate life of the pipe.

In the Northwest, some makers of wood pipe are now equipped to furnish creosoted or otherwise treated fir staves. The writer sees no reason for not advancing pipe built of fir properly treated from Class *B* to Class *A* of the author's specifications, although sufficient time has not yet elapsed to demonstrate fully either the complete correctness of this or the most economical treatment available for producing the desired result.

The proper creosoting of pipe staves is not only expensive in itself, but also adds heavily to the freight charges and, to some extent, to the cost of pipe erection. In some sections of the country such pipe appears to have competed successfully with untreated redwood.

It is also as yet an open question as to what is the life of exposed fir pipe, and to what extent its life is extended by surface coating. The only case of decay of such pipe, which has come to the writer's attention is that of a pipe built on saddles in Utah (listed as No. 16 in the writer's report), and this was after 12 years of service. This pipe was not coated, and other uncoated pipes of this kind showed no decay after 14 years; and the writer has found, in the New England States, sound stave pipe, built probably of native pine, and resting on piers, in regard to which the time of construction reached back beyond the memory of any one who could furnish information.

On the other hand, the author has noted decay in exposed redwood pipe in Southern California, maintained full only during the irrigation season, and less than 15 years old.

The author's statement regarding the superiority of air-drying over kiln-drying is undoubtedly true, so far as past commercial methods are concerned. It is understood, however, that the subject of kiln-drying has been under scientific investigation for several years, and it is not improbable that commercially feasible methods will be perfected

Mr. Henny. which will be free from the objections of wood injury to which reference was made.

So far as continuous-stave pipe is concerned, there is no serious disadvantage in the use of lumber not fully dry, and a rigid adherence to the requirement of one year's air seasoning may tend to place a serious limit on reasonable competition, or even make it impossible to obtain the material within the time required.

The author believes that the use of inserted or slip-joint pipe is not to be recommended. The only pipe of this kind examined by the writer was made of fir and, so far as the joints are concerned, was entirely successful. Pipe of this kind on the Sunnyside Project of the Reclamation Service successfully withstands a pressure of 150 ft. No injury was noted in handling or laying, contrary to the author's fears, which, however, may apply more specially to the weaker and softer redwood.

It is noted that, in the classification suggested by the author, Class *A* pipe is to have a life of 25 years or more under all conditions. On further thought he will undoubtedly desire to place some limitation on these conditions, as he must be aware of several instances of buried and exposed redwood pipe which, owing to the fact that it is not completely water-filled at all times, in combination possibly with the use of coarse-grained wood, has shown a comparatively short life.

Some 30 years ago, when continuous-stave pipe was first introduced into the Western States by the late Charles P. Allen, of Denver, Colo., it was looked on by most engineers as of doubtful utility. It has gradually established itself as a legitimate type which can be adapted to relatively permanent or temporary uses, as requirements or financial advantage may dictate.

Experience as to its life and usefulness is in fact accumulating, and renders it possible to proceed with a steadily growing sense of security as to results, to which the able exposition of fundamental principles by the author is a valuable addition.

Mr. Rust. HENRY P. RUST,* M. AM. SOC. C. E.—This is a most opportune time to advocate the use of wood in place of steel for all construction purposes, wherever possible, with due consideration for safety; and, from the title of this paper, the speaker was most hopeful that it would fill a much needed want. However, on closer examination, the paper is most disappointing, as it is evidently an attempt to advocate the use of redwood in place of Douglas fir for wood pipes, and it does not offer proper proofs for such statements as "Douglas fir and pine are fundamentally inferior to redwood."

Although, in the early days of the use of such pipe in the West, redwood was practically the only material considered, recently, Douglas fir has been more generally used. On this account the highly desirable

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* New York City.

object of such a paper will be defeated, and engineers having independent experience and realizing the circumstances will hardly treat it seriously. It will be unfortunate if the discussion turns merely into a contention between the makers of redwood and Douglas fir pipe, as is likely, because the choice of material is not the most important feature, and, under proper conditions for any wood pipe, either of these woods should be satisfactory. Mr. Rust.

There is no doubt that redwood will resist decay much better than Douglas fir or long-leaf pine, when used in any location where it will not be saturated with water; but it has always been considered a condition necessary to the successful use of wood pipe that the wood be kept saturated and the pipe full of water practically all the time. This is really not a difficult condition to meet, and will not cause the rejection of wood pipe in many cases where it might otherwise be used. If the pressure is too low or the pipe cannot be kept filled, a different construction can probably be used, which will give better satisfaction, with a comparatively lower first cost. The contention, however, that even redwood pipe should be used where the staves are not to be kept saturated practically all the time can hardly be sustained, and although it may have proved satisfactory in some such cases as the author notes, there are many other cases where it has decayed from this cause, although they have not been mentioned in the paper.

There are other causes of failure besides decay, and there are objections to the use of redwood. The wood is softer and much more brittle than pine or fir. It cannot stand as great compression, and, in consequence, such pipe is more subject to leakage, to overcome which a comparatively larger number of bands should be used.

The Great Northern Power Company has four redwood pipes, each about 3 000 ft. long. The first three were constructed about 10 years ago, and evidently an insufficient number of bands were used for this material. The pipes leak quite badly in places, and the bands have to be tightened frequently. This is a more serious matter than would at first appear, as the pipes are covered, and it is expensive to excavate the back-filling in order to tighten the bands. In fact, it is now found that merely tightening the bands will not remedy the trouble, as they sink into the soft wood so that the leakage soon becomes as bad as ever. The fourth pipe, which was built about 3 years ago, had a much larger number of bands, and thus far it has been satisfactory.

It is reported that the failure of the redwood pipe used by the City of Provo was caused by the bands under pressure sinking into the wood to such an extent as to cause very serious leakage. Recently, a number of engineers have used Douglas fir on rather large and important pipe lines, not only because they considered that where the wood was kept saturated, Douglas fir would last as long as redwood,

Mr. Rust. but also because it is stronger, will stand more distortion without leakage, and is much cheaper.

The Guanajuato Power Company has also had some trouble with one of its redwood pipes on account of the silt in the water scoring the bottom of the pipe. There has been considerable wear along the bottom staves, particularly at some of the butt joints, where a small pocket has been eaten into the staves, about one-third of the way through. This pipe has been in use for about 10 years.

One of the largest users of wood pipe is the Denver Union Water Company, which has about 100 miles, ranging from 24 to 60 in. in diameter. The Company's first large pipe, laid in 1890, is of long-leaf pine, and is still giving good service. The next pipe, laid a couple of years later, is partly of pine and partly of redwood. Since that time, Douglas fir has been used in all the wood pipe built by the Company, and it has evidently been found very satisfactory.

There are also various examples of pine pipe in the East, which have proved satisfactory, one of which might be mentioned. The D. E. Converse Company, at Glendale, S. C., has a 78-in. pipe made of short-leaf pine, which has been in use for 26 years. There is back-filling around the lower half of the pipe, and the pressure is comparatively light, only a few feet head. Although some decay has taken place on the outside of the staves, the pipe is still in use and still satisfactory.

The author speaks of fir and pine being coarse-grained, but such wood is hardly suitable for pipe, and is not used where the specifications are properly drawn and the material properly inspected. In fact, there is now a specification covering the density of fir, which it is to be hoped will be generally adopted. Fir and pine staves have not been made any thicker than those of redwood, and have been quite satisfactory under high pressure.

A large quantity of kiln-dried fir has been used for pipe, and has proved satisfactory. No doubt kiln drying does make redwood very brittle; it should be air-dried. This, however, is a disadvantage of redwood rather than of fir.

The speaker has no desire to advocate the use of fir in preference to redwood, as he considers the latter equally satisfactory under proper conditions. However, it would be a mistake to create an unfair prejudice in favor of one as against the other.

There has been considerable discussion lately as to whether wood pipe should be buried or left exposed, and as to whether the outside should or should not be painted. There is no doubt that a pipe should be buried completely at least 2 ft. below the surface, or left entirely exposed. However, in the latter case, the necessary supports are a matter for serious consideration, especially for pipes of large diameter on soft ground.

In a dry climate, where a pipe is buried in a dry, sandy, or pervious soil, the ground will absorb the moisture from the outer shell of the wood, so that it cannot be kept saturated, and this causes a condition favorable to decay. Painting the outside of any pipe under such conditions would be most advisable, but if the pipe is laid in wet soil, where the ground itself is always saturated, it does not seem logical to paint the wood. Also, it is advisable to keep the pipe away from all vegetable matter and growing plants, the roots of which will absorb moisture from any wood and cause it to decay. Mr. Rust.

Creosoting may be advisable to meet some of these special conditions. It is now being used to some extent, and creosoted fir under ordinary conditions costs only a little more than redwood.

As to the statement that "a coating, to be effective, should be applied diligently and often", it has generally been considered necessary to coat a pipe only where it is buried, in which case, of course, it is impossible to paint it often.

Regarding the standard specifications for bands submitted by the author, as stated, it is quite common practice to use a factor of safety of 4 against breaking due to tension caused by the hydrostatic pressure. However, this does not allow anything for the additional tension required to keep the staves tight and prevent leakage; and a factor of safety of 5 should be used, or some additional allowance should be made for the extra tension required for tightness.

A working stress of 800 lb. per sq. in. for the bearing strength of the wood is excessive. It has been found that one of the causes of decay is the bruising of the wood through excessive pressure on the bands. A working stress of 400 lb. per sq. in. would probably be more satisfactory, and would not require the addition of any large number of bands, except in the cases of pipes of small diameter. It hardly seems reasonable to use a larger working stress in this case than that usually considered good practice for other structures. The experience of the Great Northern Power Company, already mentioned, bears out this contention.

Regarding the thickness of the staves in Table 2, there is a jump from 2½ to 3½ in. at a diameter of 6 ft. The Douglas fir manufacturers have an intermediate standard thickness of 3 in., which can be used for pipes from 6 ft. up to 9 or 10 ft. in diameter. This has been quite satisfactory, and makes considerable saving in the quantity of wood required for pipes between these sizes. There seems to be no reason for not using an intermediate standard thickness of 3 in. with redwood.

Regarding the author's proposed specifications dividing wood pipe into three classes, *A*, *B*, and *C*, according to the wood used, apparently the only reason for this is to have an opportunity of putting redwood in a supposedly much superior class by itself, which, of course, would

Mr. Rust. give the manufacturer of redwood pipe a good excuse for asking higher prices for his product.

It would be much more logical to vary the thickness of the staves, the band spacing, and other details which have more effect on the cost of a pipe, and to have three tables such as Table 2, one for each of the different classes. Otherwise, the specifications are those generally proposed by the manufacturers of redwood pipe.

As stated by the author, there is no reason for not making the specifications for fir as strict as those for redwood. However, this is the fault of the lumber mills. The proprietors of Douglas fir mills have not shown themselves to be very broad-minded in many ways, and have done nothing to encourage the use of wood pipe.

It would be of considerable advantage if the wood stave pipe manufacturers had an association, organized on the same lines as the Portland Cement Manufacturers' Association, which would be in a position to give disinterested and authoritative data regarding wood pipe on which engineers could depend; and it would result, no doubt, in the use of a much larger quantity. It would also be of considerable help to engineers if manufacturers would keep a stock of pipe or pipe material on hand at some point in the East, so that in case of emergency it would not be necessary to wait a month or two in order to obtain material from the West.

Wood stave pipe has been in general use in comparatively important works for about 30 years. This should be a sufficient length of time for engineers to judge of its permanence, the precautions to be taken with it under different conditions, and to determine what constitutes a safe design. However, as the author states, many engineers in the East still regard such pipe with suspicion, owing chiefly to reported failures which, when investigated, show either that wood pipe should never have been used, or that proper precautions had not been taken to meet the special conditions. There has been ample experience with such pipe, and much information has been published regarding it. It is now possible to find out what constitutes good practice, and, of late, there have not been as many failures reported as in the early years of its use. At the present prices, wood stave pipe costs only about one-third or one-fifth as much as steel pipe, under conditions where either could be used.

Mr. Robbins. F. M. Robbins,* Esq.—It has been suggested that there is a lack of co-operation among the wood stave pipe manufacturers. This is true; and it has held back the development to which good wood pipe is entitled. A closer association would result in the elimination of the poorer pipe which causes most of the unpleasant notoriety. It is not a question of redwood or fir, but of proper design and construction,

* Secy., The Marion Malleable Iron Works, Marion, Ind.

as investigation shows that most failures in wood pipe are due to design by inexperienced engineers, or construction by outside contractors. A standard specification for wood stave pipe is urgently needed to guide the engineer and eliminate the contractor who builds on a price basis only in keen competition with the established pipe builders. Mr. Robbins.

As stated in the paper, there is "provision for rigid supervision of the bands, though adequate stress is not laid on the requirements for the shoes, which, after all, must be capable of developing the full strength of the band."

The cases cited are only a few of those in which shoes were used which developed only partly the strength of the bands. The cost of the shoe is insignificant as compared with that of the band, and yet its weight, on lines where price competition is keen, is frequently reduced to a point where the value of the steel wasted amounts to several times that of the shoes, and the pipe banding is less than half as efficient.

Furthermore, tons of excess weight have been buried where the shoes were heavier than needed, or where the metal in them was not properly distributed.

Standard lines of properly designed and thoroughly tested shoes have been developed, and these can be used with safety where the specifications are rigid enough to admit no variation or substitution by the contractor.

The specification for pipe shoes advanced by Mr. Partridge is much better than those heretofore offered, but it is not yet as rigid as it should be. The advance in malleable iron has made it possible to secure an even better quality, and the specification should read: "Shall have a minimum tensile strength of 45 000 lb. per sq. in., and a minimum elongation of 7½% measured in 2 in."

The strength of the shoe should be stated definitely as the elastic limit, there having been cases where this was evaded by using the ultimate strength of the shoe as compared to the elastic limit of the band.

The coating of malleable iron is a question that will bear further investigation, as, generally speaking, malleable iron does not corrode, to an appreciable extent, after the first or surface oxidation. The coatings now applied are unnecessary in ordinary circumstances; and, under abnormal conditions, galvanizing or coating with red lead should be recommended.

WILLIAM J. BOUCHER,* Assoc. M. Am. Soc. C. E.—The use of wood stave pipe is somewhat limited in the East, but a pipe of this material has been completed recently for the water supply of Watervliet, N. Y., under the direction of G. R. Solomon and P. H. Norcross, Members, Am. Soc. C. E. In New York City, however, there is a type of stave pipe which is hidden from the public view so completely Mr. Boucher.

* New York City.

Mr. Boucher. that comparatively few are aware of its existence. Reference is made to the sewer outlet barrel pipes built under the piers on the North and East River fronts. As a sewer outlet at the bulkhead wall would be very objectionable, all sewers are extended to the pier head line by constructing the wood barrel for the full length of the pier at the foot of the street in which the sewer is built. These wooden pipes have been the subject of special study by the Bureau of Sewers of New York City for some time, and are now standardized.

Of course, such pipes need not be designed for the high pressures encountered in water supply systems, as the sewers seldom flow full. They are built of creosoted lumber in order to protect them from the effects of alternate wetting and drying, due to the rise and fall of the tides. The invert is frequently at or near mean low water (in order to obtain as much fall as possible), and this causes the upper portions of the barrel to be submerged only for the short period of high tide. Fig. 1 shows the design for one of the most recent sewers.

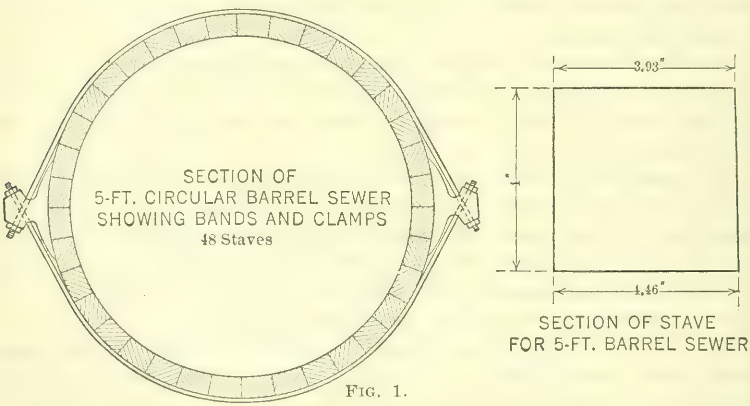

Fig. 1.

The following are the standard specifications of the New York City Bureau of Sewers:

Specifications for Wooden Barrel Sewer.

"Wooden barrel sewers shall be built of creosoted wooden staves held in place by galvanized metal bands, and supported by timber framework.

"All the timber used in the construction of the barrel staves shall be sound commercial short-leaf yellow pine, free from the following defects: large, loose, unsound or hollow knots, through shakes or round shakes, worm holes and knot holes. All pieces shall show, either, one heart face or two-thirds heart on both sides.

"Timber required for the permanent support of the sewer shall be long-leaf yellow pine of the quality known as 'Merchantable,' graded

in accordance with the rules known as the 'Inter-state Rules of 1905, Adopted by the New York Lumber Trade Association.' Mr. Boucher.

"The barrel staves shall be accurately milled on all sides to exact shape, so that they will form a sewer with tight joints true to the required dimensions.

"The Contractor shall make such tests of the materials and samples as required, and afford every facility requested for measuring tanks, cylinders, gauges, etc., and for taking and analyzing samples as often as may be deemed necessary, including the use of a properly equipped laboratory and other necessary apparatus. The manufacturer shall equip his plant with all necessary gauges, appliances, and facilities to demonstrate that the requirements of the specifications are being fulfilled.

"Upon notice from the Contractor that it is ready for inspection, all timber and staves (after the staves have been milled) shall be examined at the one time prior to creosoting. Rejected timber shall be separated from that approved, and the approved timber only shall be creosoted. The Contractor shall give ample notice to the Engineer so that he may arrange for inspecting the creosoting at the place of manufacture.

"The creosoting of the timber shall consist of two operations:

"*a.*—The preliminary application of steam and vacuum.

"*b.*—The injection of a minimum average of 18 lb. of oil into each cubic foot of timber. In addition to this minimum average, additional oil shall be injected into the timber, depending upon the physical condition of the timber, sufficient to render it possible to meet the 5% absorption test specified herein.

"The preservative to be used shall be a distillate of coal gas or coke-oven tar, and shall be free from all adulteration and contain no raw tar, filtered or unfiltered tars, or pitches, petroleum compound, or other tar products.

"It shall be completely liquid at 38° cent. and shall have a specific gravity at that temperature of not less than 1.03 nor more than 1.08.

"It shall contain not more than 2% of matter insoluble by hot extraction with benzol and chloroform.

"On distillation, which shall be made according to the Borough President's standard method of tests, the description of which is on file in the office of the Engineer, the distillate, based on water free from oil, shall be within the following limits:

"At 210° cent., not more than 5 per cent.
At 235° cent., not more than 38 per cent.
At 315° cent., not more than 85 per cent.

"The distillate between 210° cent. and 235° cent., shall yield solids on cooling to 15° cent. The preservative shall contain not more than 3% of water.

"Samples of the preservative taken from the treating tank during treatment shall at no time show an accumulation of more than 2% of saw-dust, dirt, or other foreign matter. Due allowance shall be made for such accumulation of foreign matter by injecting an additional quantity into the blocks.

Mr. Boucher.

"Before being treated, the timber shall be air-dried to such an extent that its weight per cubic foot does not exceed 50 lb.

"Live steam shall be admitted into the cylinder, and applied to the timber, and gradually raised during a period of 1 hour, to a boiler gauge pressure of 15 lb. at 185° Fahr., and maintained for a period of 2 hours for the timbers weighing 40 lb. per cu. ft.; or it shall be gradually raised during a period of 2 hours to a boiler gauge pressure of 25 lb. at 220° Fahr., and maintained for 5 hours for the timbers weighing 50 lb. per cu. ft.; then a vacuum of about 23 in. shall be applied for $1\frac{1}{2}$ hours for the timbers weighing 40 lb. per cu. ft., and for $2\frac{1}{2}$ hours for the timbers weighing 50 lb. per cu. ft., with the temperature of the cylinder maintained at 150° Fahr. For intermediate weights of timber the above treatment shall be varied as directed. Timbers whose average weight per cubic foot varies by more than 5 lb. shall not be treated in the same charge.

"Oil at not less than 180°, nor more than 190°, shall then be admitted into the cylinder, and the pressure gradually raised during a period of 3 hours to 165 lb., or until an average minimum of at least 18 lb. of oil per cu. ft. has been forced into the timber, and, if necessary, an amount in addition thereto sufficient to render it possible to meet the 5% absorption test specified herein. During this period the temperature of the oil shall not be allowed to fall below 165° Fahr. The free oil shall then be expelled from the cylinder.

"After treatment, the timber shall be held in the cylinder for about 1 hour, and shall then be withdrawn. The treated timber shall be protected from the direct rays of the sun, and shall be loaded for delivery within 48 hours after withdrawal from the cylinder.

"After delivery, but before placing in the structure, the timber shall show such water-proof qualities that, after being dried in an oven at a temperature of 100° Fahr., for a period of 24 hours, weighed, and then immersed in clean water for a period of 24 hours, and again weighed, the gain in weight shall not exceed 5 per cent.

"The barrel sewer shall be built so as to form a continuous structure, unless otherwise directed. If built as a continuous structure, each stave shall be placed so as to form a lap of at least 4 ft. with the adjoining stave. Each stave, as put in place, shall be nailed to the adjoining stave, in place, with tinned-wire nails 7 in. long, spaced at intervals not exceeding 3 ft. Staves shall be thoroughly fastened to each other and to the frame and chocks, and shall be not less than 20 ft. in length, except at closures.

"All joining, framing, and mortising shall be done in a workmanlike manner. Holes, of the sizes required, shall be bored for all spikes and bolts.

"The acceptance of material at the plant of manufacturer is tentative only, and the City reserves the right to reject shipments, in whole or in part, after delivery on the line of work, if the material fails to comply with the requirements of the plans and specifications.

"Galvanized wrought-iron or steel bands, saddles, hangers, manhole covers and frames, angles, bolts, washers, etc., including all metal required in the permanent structure, shall be furnished, placed,

and adjusted as shown on the plans and in accordance with the requirements of these specifications. Mr. Boucher.

"The steel and wrought iron shall comply with the requirements, and be subject to the tests provided in the standard specifications most recently adopted by the American Society for Testing Materials.

"The bands, saddles, hangers, manhole covers and frames, shall be brought to true dimension and shape indicated on the plans before galvanizing. The extra thickness of the band shall be obtained by 'upsetting' the ends. Allowance shall be made in threaded articles so that the parts may screw together after galvanizing without recutting the threads. Samples of the metal to be used in the structure shall be submitted for test and approval before galvanizing. Approved metal shall be galvanized as follows: the iron and steel surfaces must first be cleaned of all scale and rust by means of steel brushes and a dilute sulphuric acid bath. When cleaned, the metal shall be plunged into a bath of molten zinc covered with sal-ammoniac.

"The galvanizing must show an even distribution of zinc over the entire surface of the steel or iron, bright in color, and not blotchy in appearance. At least 1 lb. of zinc shall be applied to every 6 sq. ft. of surface."

J. C. RALSTON,* M. AM. SOC. C. E. (by letter).†—A more or less continuous experience of nearly 18 years in the use, design, and operation of wood stave pipe, and a fairly intimate knowledge of manufacturing methods, during which time the writer has had to do, in a consulting capacity, with more than 3 840 miles of all sizes, including more than 40 miles of continuous stave pipe of an average diameter of 70 in., impels him to discuss briefly certain phases of Mr. Partridge's paper, and to call attention to some incorrect statements and immature conclusions. Mr. Ralston.

The author says, on page 568,‡

"In machine-banded pipe there is absolutely no basis for the present-day so-called specifications."

This needs no comment other than to add that the specifications suggested in the Appendix are not equal to those of the best practice. The basis, foundation, groundwork, or controlling principle on which the author's or any other engineer's specifications for wood pipe are drawn, is the same, namely, the need for a wood pipe.

The evolution of the wood pipe specifications has passed through a long period, and has engaged the efforts of such distinguished engineers as the late J. D. Schuyler, J. T. Fanning, A. L. Adams, Members, Am. Soc. C. E., and others, not to mention any of the present-day lesser lights, and it is known that they found a very substantial and satisfactory basis for both the specifications and the pipe.

* Spokane, Wash.

† Received by the Secretary, July 5th, 1917.

‡ *Proceedings*, Am. Soc. C. E., April, 1917.

Mr. Ralston.

Again, in reference to fir (yellow pine, Oregon pine), on page 577,* the author states that the oldest lines have been built not more than 10 years.

It is useless to encumber the record with a long list. Two or three citations may be permissible: The Garoga River Project, in Fulton County, New York, is noteworthy. This power plant was built in 1850, and served until a fire destroyed the mill in 1903. The original project comprised a triangular timber crib dam (Fig. 2), about 15 ft. high, much the same as those of the simpler types constructed by Mr. James F. Smith, Chief Engineer, in 1818 and 1819, for the Schuylkill Navigation Company—structures which served from 40 to 50 years. The dam was connected to the mill about 600 ft. down stream by a 30-in. wood stave pipe, through which the water was delivered to an overshot wheel about 10 ft. in diameter. The pipe staves were of hard pine, 2-in. scantling, jack-planed on their edges to the proper bevel, and then assembled. The bands or hoops were of 2-in., 16-gauge, old-process, puddled iron, and were as well preserved as the staves when the line was put out of commission. Angles in the pipe line were turned through wood stave pipe wells, after the manner of a sewer. The line has fallen into decay since the mill was burned in 1903.

Aside from the historical engineering interest of such a well-designed piece of work, as well as its significance as a forerunner of the hydraulic end of the modern power plant, the three outstanding features of interest at present are that the staves were of hard pine, that they were in substantially as good a state of preservation as the iron bands at the end of 50 years, and that the line had a life so much longer than 10 years.

In 1913, a new power project replaced this primitive plant, and includes about 2 miles of 78-in. wood stave pipe under a maximum head of 160 ft. (Fig. 3.) After an extensive inquiry into the merits of various woods, Douglas fir was selected for this pipe.

Another of the pioneer lines in wood stave pipe construction is that built by the late Mr. Fanning, at Manchester, N. H., in 1874. This was a buried line, and was put in operation in the spring of that year. It continued in operation for 40 years, and was finally displaced, not because the pipe was in bad condition, but because a new power installation with a higher head and a new location had to be chosen. Mr. Fanning's line was 72 in. in diameter, and was of 4-in. "hard pine staves * * * cut from pitch-pine plank."†

Here, again, is another notable example of hard pitch-pine staves serving for four decades instead of one.

* *Proceedings*, Am. Soc. C. E., April, 1917.

† *Transactions*, Am. Soc. C. E., Vol. VI, p. 69.

Fig. 2.—Dam and Intake of Garoga River Plant Eight Years After it Went Out of Commission.

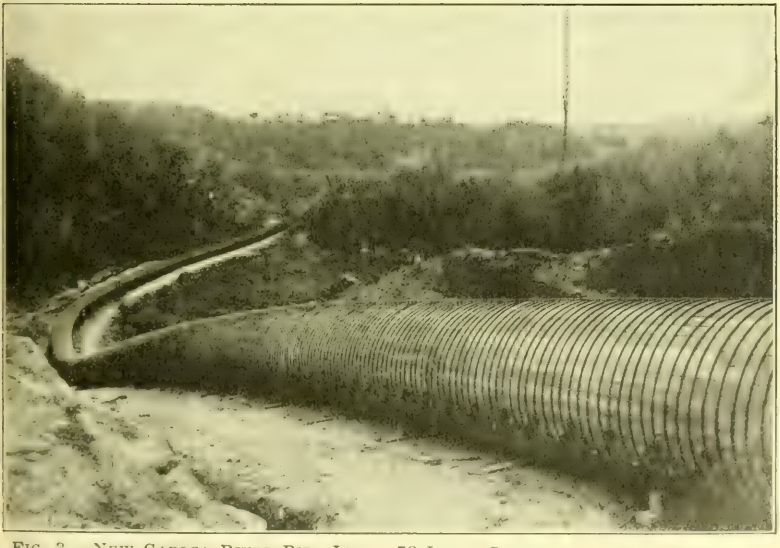

Fig. 3.—New Garoga River Pipe Line: 78-Inch, Continuous-Stave Line, of Douglas Fir.

Mr. Ralston.

Another classic example is the Ogden pipe line. It contains 27 000 ft. of Douglas fir stave pipe, 72 in. in diameter, under a head of from 55 to 117 ft., and was constructed in 1896.

Henry Goldmark, M. Am. Soc. C. E., describing this line, stated, among other things, "A new departure, too, is the use of Douglas fir in place of California redwood. The former timber is much harder and stiffer."* D. C. Henny, M. Am. Soc. C. E., in the ensuing discussion, remarked, "The use of Douglas fir in stave-pipe construction can hardly be called a new departure, as almost all the stave pipes built in Washington, Oregon, and British Columbia were constructed of that material." The late Arthur L. Adams, M. Am. Soc. C. E., in a continuance of the same discussion, added, "The author [Mr. Goldmark] is mistaken in thinking the stave pipe [Ogden line] a pioneer, either in its diameter or in the use of Douglas fir for staves. Several conduits 6 ft. in diameter have previously been built, while the same quality of timber has been repeatedly employed."

Here, then, are prominent engineers engaged in discussing this type of pipe with Douglas fir staves, and not redwood, 20 years ago; and, later, in the U. S. Forest Service *Bulletins*, there are references to Douglas fir or Oregon pine as "the most important of American woods", and "as a structural timber it is not surpassed."

The writer presumes to assert that had the author known more history, and had he been familiar with woods other than redwood, he hardly would have made the following misleading statements:†

"In all the discussions it is noticeable that there are no references to cases where wood stave pipe has been constructed or operated successfully, or unsuccessfully, for a number of years. Actual cases where it has been in service long enough to give an idea of its durability are wanted by engineers."

* * * * *

"Fir is the pipe that has failed, the oldest lines having been built not more than 10 years * * *."

Yet, at the same time, he admits that "Fir and pine are pitchy woods." The inescapable but (the writer is persuaded) unintentional paradox by the author is that pitchy woods are long-lived.

The superior physical and mechanical qualities of Douglas fir are now well known to the Profession, and it is unjust, misleading, and not in the interest of scientific truth for the author to state directly and by repeated implications that Douglas fir is inferior to redwood. Both are excellent woods, the best known for stave material. Although the writer's experience of 18 years in stave pipe construction definitely commits him to Douglas fir, he would not belittle the excellent qualities of redwood. If he did, he could cite examples

* *Transactions*, Am. Soc. C. E., Vol. XXXVIII, pp. 268, 311-312.

† *Proceedings*, Am. Soc. C. E., April, 1917, pp. 561, 577.

Mr. Ralston. wherein redwood lines have been repaired with Douglas fir staves. Doubtless, instances might also be cited involving the opposite practice.

Side by side and under identical conditions, one wood will last as long as the other, except in the case of continuous stave lines, when Douglas fir will take the premier position. Not only is this the writer's mature judgment, but the latest practice of some of the leading engineering corporations, and some of the most eminent consulting engineers confirm the latter part of this statement. Thus, the two largest continuous stave lines in the West, 162 and 168 in. in diameter, respectively, Figs. 4 and 5, and the largest in the Atlantic States, 144 in. in diameter, two of which are for commercial plants and one for railroad electrification, built within the last 3 years, are of Douglas fir. One of the Western lines is buried, the others are above ground.

In the matter of the wire-wound type, from 2 to 24 in. in diameter, used in domestic water supplies, the greatest annoyance has arisen in cases where the purchaser has insisted on the cheapest possible pipe, with sap staves and light wire, regardless of the assurance that heavier wire and selected, sapless staves would give better and longer service. In the course of time—the purchaser having been displaced by a new operator and the purchaser's demands forgotten—criticism, and sometimes condemnation, of all wood pipe has resulted.

It is true, as the author suggests, that the nature of the back-fill often has much to do with the life of the pipe. Careless or ignorant methods in laying the pipe and in back-filling the trenches are the cause of trouble and reduced life. The most important consideration, next to the quality of the pipe, is the selection of clean earth back-fill devoid of vegetable mould, roots, grass, sod, or other unstable material, as well as its proper compaction, preferably for the full depth of the trench. Percolating swamp waters are very destructive. Their carbon dioxide is the arch enemy of wire, rods, and steel pipe, although not particularly harmful to the stave wood. The life of a buried pipe is always increased materially when percolating waters, free or entrained oxygen, leaching processes, or the formation of alkaline salts are excluded. The writer has examined municipal pipe lines which had been laid for 10, 12, and 14 years under compacted clay back-fill, and found the galvanizing on the wire as bright as when it left the factory; and the tar coating on the stave had to be scraped away, in order to reach the wood, which was in perfect condition. An indefinite life could almost be ascribed to such lines.

The writer can well understand how such careful methods of back-filling might largely account for the well-preserved specimens of wood pipe that have been exhumed, from time to time, in London

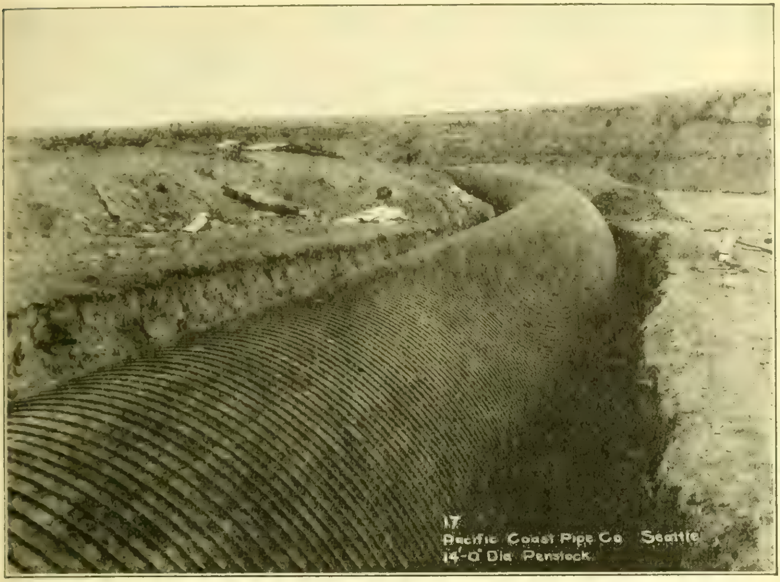

FIG. 4.—FOURTEEN-FOOT LINE OF MONTANA POWER COMPANY, NEAR GREAT FALLS, MONT., OF DOUGLAS FIR.

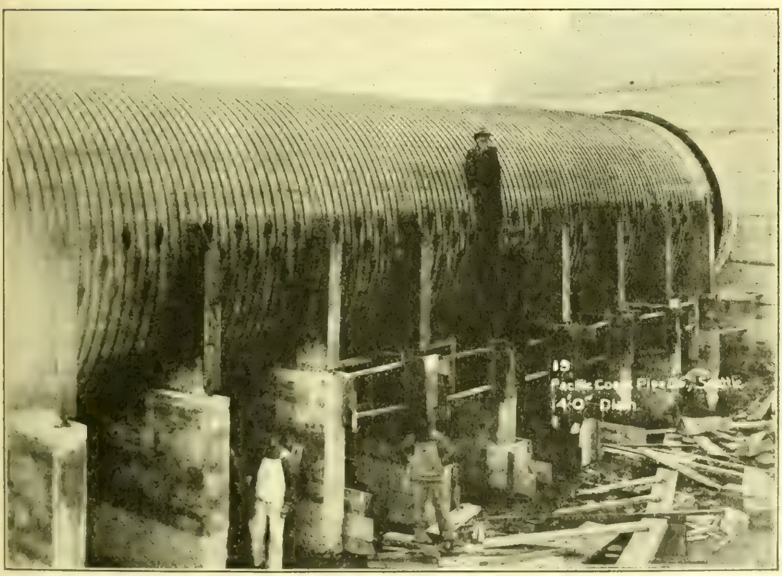

FIG. 5.—ANOTHER VIEW OF 14-FOOT LINE OF MONTANA POWER COMPANY.

and some of the older American cities, which had been buried from one to two centuries. Mr. Ralston.

On the other hand, the writer has examined lines, the porous back-fill of which had been percolated and partly washed away by continuously flowing swamp water, and has found that the wire banding had been destroyed in 3 years. Thus, there are other elements than the selection of good materials, or the manufacturer's technology, which profoundly affect the life of wood stave pipe.

The use of wood stave pipe and the science of its construction, when left in the hands of experienced engineers, now rests on a sound, intelligent, and definite economic basis. Its use is no longer experimental or a makeshift. Neither are its design, materials, manufacture, or erection invested with the uncertainties ascribed by the author. It is true that in wood pipe, as in nearly all classes of engineering products, poor materials and workmanship are sometimes found, but that does not condemn the type, nor is the technology of the subject so impoverished that it can be metamorphosed by the one-sided specifications suggested by the author.

Except in the most obvious cases of temporary service, there should be only one standard of excellence, namely, the best. The variables in dimensions, strength, and materials are fixed by service demands, and are well understood and simple.

Contrary to the author's statements, the quality and kind of material in the staves are well defined in the manufacturers' and in the latest Reclamation Service specifications. The following is quoted from Specifications No. 73-D, U. S. Reclamation Service, Shoshone Project, Wyoming, dated March 26th, 1917, and may be taken as representative of good practice:

"All lumber used in staves shall be Douglas fir. It shall be sound, straight-grained, and free from sap, dry rot, checks, wind shakes, wane, and other imperfections that may impair its strength or durability. Pitch seams will be permitted in not more than 10% of the total number of pieces, it showing on the edge only, and if not longer than 4 in. nor wider than $\frac{1}{16}$ in.; no through knots or knots at edge or within 6 in. of ends of staves will be allowed; sound knots not exceeding $\frac{1}{2}$ in. in diameter, not falling within the above limitations, not exceeding three in number within a 10-ft. length, will be accepted. All lumber used shall be seasoned by not less than 60 days air drying in open piles before milling, or by thorough kiln drying. All staves shall have smooth planed surfaces, and the inside and outside faces shall be accurately milled to the required circular arcs."

The foregoing fixes the quality and kind of material. The thickness of the finished stave will depend on the diameter of the pipe and the pressure under which it serves. Stave thickness varies from

Mr. Ralston. 1 in. in the smallest machine-banded pipe to 4 in. in the largest continuous stave pipe.

The paper and suggested specifications ineptly seek to give the impression that all first-class stave pipe should be made only from clear, one-year, air-dried redwood, and that only the inferior grades of pipe may be made of Douglas fir. This is noticeably conspicuous in the suggested "Specifications for Continuous-Stave Pipe, Class *A*," and the "Specifications for Machine-Banded Pipe, Class *A*." In both these Class *A* types, the writer quite agrees with the author that, where redwood is used, it should be a clear, air-dried product, for the reason, as seems implied, and as is indeed true, that kiln-dried redwood is brittle and structurally inferior. The writer's best judgment and experience, however, are (and a marked preponderance of the best practice throughout the country confirms him) that Douglas fir, either air- or kiln-dried, should also have been specified, unless it was the author's purpose to write exclusively a redwood specification. If, however, he desired to write a standard, Class *A*, wood stave pipe specification, then, in the interests of engineering fact, practice, and non-partisan purpose, Douglas fir should not have been omitted. If redwood is to be substituted for Douglas fir, then, considering equivalent strength, if the safety factor is closely involved, as well as the greater porosity of the former wood, the thickness of the redwood stave should be correspondingly increased.

It is granted that the grain in redwood is not so coarse as in Douglas fir, but the mechanical ultimates of redwood are less than in fir; yet it should be noted that the soft or winter-growth portion of each annual ring in redwood is much more porous than the similar soft portion of each annual ring in Douglas fir. Nevertheless, the whole question of percolation and saturation in either material is largely academic, if not fanciful. Both woods, or any wood, will become saturated and remain so, as long as the pipes are full of water under a sensible pressure. The best practice now fixes the thickness of the stave as such that, when the requirements of saturation and strength are satisfied, the question of percolation automatically disappears. Any decrease in the standard thickness of stave, either in fir or redwood, should not be permitted.

The writer has never seen nor heard of a case of "excessive percolation" in any pipe, except where the staves were thinner than standard practice. Some redwood machine-banded pipe has been manufactured with unusually thin staves, but it is believed this practice is questionable and should be discouraged.

It is only in the case of intermittent service, such as in irrigation lines, where saturation becomes important. Here, happily, a definite solution of the problem is at hand at a slightly increased cost, that

is, to creosote the staves, both fir and redwood, just as the best practice now requires that the wooden sleeves of wire-wound standard pipe shall be creosoted, because of a less perfect saturation in the sleeve than in the pipe. Mr. Ralston.

The writer partly agrees with the author when, on page 577,* he says:

"The greater number of failures possibly originated at the joints. The outer edges of the staves in the collars, when wood collars were used, decayed rapidly owing to the fact that fir needs saturation for preservation, and saturation was not secured at those places."

If the author's diction had been more precise, and he had said that the ends of the staves in the collars decayed more rapidly than the walls of the pipe, owing to the fact that all woods need saturation to secure the best preservation, then the writer would agree with him. Redwood does not enjoy a special dispensation of immunity from decay. Metal collars are not a success, and are more or less obsolete. The open-tank creosoted wood collar has proved superior and satisfactory. It should be borne in mind, however, that when the entire pipe is to be creosoted, the process of creosoting must then be with the closed-tank, pressure method, and that the average oil content must be not less than 8 lb. per net cu. ft. of wood. The open-tank or dipping method for collars is used successfully where the pipe proper is not creosoted. The reason is obvious. Clear kiln-dried Douglas fir, when given the open-tank treatment, will get a thorough penetration in the end grain to a substantial depth. The penetration into the side of the stave will vary, depending on the nature of the lumber and the location of the annual rings in relation to the surface of the stave. Collar troubles arise from decay that sets in on the ends of the staves and works back, finally undermining the wires. If the ends of the staves in the collars are protected by open-tank treatment, from 90 to 95% of the protection afforded by pressure treatment is secured at a greatly reduced cost. Pressure creosoting, fortunately, permits the use of sapwood in staves, and where such method is used, the specifications should be correspondingly changed. The writer believes that properly creosoted wood pipe for high-class permanent work will meet with great favor, and has an encouraging future.

An instructive series of tests was recently made by the Engineering Department of the West Coast Lumbermens' Association, in co-operation with the University of Washington, on twenty-three Douglas fir staves, taken at random, which had been in continuous service for 16 years on the Cedar River pipe line of the City of Seattle. On the sections of pipe examined, and from which the staves were taken, "there was not a single stave in either pipe section which showed

* *Proceedings*, Am. Soc. C. E., April, 1917.

Mr. Ralston. any signs of decay." The specimens were tested in compression parallel to the grain to determine their crushing strength. The staves had been under 22 lb. hydraulic pressure for that length of time. They gave an average dry weight of 30.8 lb., and a maximum average crushing strength of 3 870 lb. per sq. in. The size of the average specimen taken from each stave was 1.63 by 5.55 in. The average number of rings per inch was 18.

Douglas fir, having a weight of 30.8 lb. per cu. ft., may be expected to have a crushing strength of approximately 4 400 lb. per sq. in.* If the original strength of the staves fulfilled this expectation, then, after 16 years of continuous saturation at a pressure of 50 ft., they exhibited 88% of the strength of new fir. Inasmuch as the mechanical ultimates of Douglas fir, with the usual safety factor of 4, are never reached, except when staves are made thinner than required by standard practice, this 12% reduction in strength is of no consequence in fir stave stock.

In the matter of curing the stave stock, extensive tests, as well as experience, have shown that the resultant difference between air-dried and properly kiln-dried Douglas fir staves is altogether negligible. In fact, there are some features of kiln-drying more favorable than air-drying, notably among others, is the tendency which this process has to develop the full extent of pitch pockets more visibly than by air-drying, whereby such defects are more easily detected by the inspector, thus making the culling process more certain. At least one of the leading Pacific Coast pipe factories has developed a special kiln method which seems to give excellent results. The ordinary dry-kilns of the lumber companies, and the methods of kilning by such companies are not suitable to the proper curing of pipe staves. Special kilns and their proper operation are the sole factors in the curing of Douglas fir stave stock, in order to make it as good as if air-cured.

It is proper, and no doubt customary, to give all wire-wound pipe, both fir and redwood, a bath in hot tar or asphalt before it leaves the factory, and, while the coating is still hot, to roll the pipe over a bed of saw-dust. This treatment is a preservative, and adds to the life of any pipe; but, even if it had no preservative quality, its value as a protection against abrasions, rough handling, and the wear and tear incident to transit, is a wise and time-honored practice. The author's contention that such treatment "increases the weight materially" is misleading, as it does not generally affect the shipping charges. The fact is that, in most cases of the standard freight car, the total weight of a load of pipe is less than the minimum car-load weight prescribed by the railroads.

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* *Bulletin 88*, U. S. Forest Service, p. 26.

Although continuous stave pipe is a comparatively simple structure, it is a mistake to assume that it can be assembled or laid without the use of expert and experienced supervision, preferably by the manufacturer's engineer, if he is an experienced man. George L. Watson, Assoc. M. Am. Soc. C. E., concludes an excellent article* with a warning to all general contractors to keep hands off, and leave all wood pipe to a regular pipe contractor. The writer is familiar with several large jobs, partly completed by local or general contractors, which had to be pulled down and relaid by an expert. Mr. Ralston.

On page 565,† we read: "Fir and pine are pitchy woods, and it is impossible to obtain commercial run lumber without sap, pitch, pitch seams, pitch pockets, and knots." The Shoshone Project specification, which the writer has quoted in this discussion as being representative, and which he recommends, does not call for "commercial run lumber," but for a stock of definite and select quality. He has never known of commercial run lumber, either fir or redwood, being used for first-class pipe.

That part of the author's suggested specification for Class *A*, Continuous-Stave Pipe, relating to redwood stave stock is not specific enough, and allows too much latitude in the character of the material. Redwood, like fir, should be straight-grained, devoid of checks and wanes, and should have no knots at the edges of the staves, nor within 6 in. of the ends, nor any defects which may impair its strength or durability. (The expression, "materially impair" should not be used.) Moreover, the width of the metallic tongue should be 2 in., because of the alleged tendency of redwood to shrink endwise.

Finally, the basis of classification, given on page 583,† in terms of longevity, seems to be more pedantic than empiric, and, therefore, should have small place at this time.

* *Journal*, Am. Soc. of Eng. Contractors, Vol. IV, No. 7, September, 1912.

† *Proceedings*, Am. Soc. C. E., April, 1917.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

DISCUSSION ON FINAL REPORT OF THE SPECIAL COMMITTEE ON CONCRETE AND REINFORCED CONCRETE*

By Messrs. C. A. P. Turner, F. E. Turneaure, and A. N. Talbot.

C. A. P. Turner,† M. Am. Soc. C. E. (by letter).‡—Mr. Scott§ analyzes the report of the Joint Committee in respect to the modulus of elasticity of steel embedded in concrete beams on the assumption that the steel furnishes the entire tensile resistance due to the moment of the applied load. This assumption is not correct. Furthermore, the assumption that the tensile strength of the concrete furnishes that portion of the tensile resistance necessary to produce a static balance of the moments not provided for by the steel is likewise incorrect. Mr. Turner.

This latter assumption, although incorrect, is found in almost every treatise on reinforced concrete published either in America or abroad. There is one treatise, however, that by Charles F. Marsh, M. Am. Soc. C. E., which shows that the distribution of stress in a reinforced beam is not analogous, or even approximately analogous, to that of a homogeneous beam in respect to any uniform distribution of intensity of the stress transversely of the beam between reinforcing bars. All agree that there is a co-action between the metal and the concrete in a beam, brought about by the resistance of bond stress, which prevents the steel from slipping in the concrete matrix. Under uniform load in a simple beam, all agree that bond stress is zero at mid-span and increases toward the end of the beam. Now, this bond force or bond stress is a horizontal shearing stress along

* Continued from May, 1917, *Proceedings*.

† Minneapolis, Minn.

‡ Received by the Secretary, April 30th, 1917.

§ *Proceedings*, Am. Soc. C. E., March, 1917.

Mr. Turner.

the surface of the steel. Consequently, it sets up tensions and compressions at 45° to the surface of the metal; the intensity of these indirect stresses is greatest next to the surface of the bar, and diminishes, according to the law of distribution of force through mass, with the distance away from the surface. The deformation of the concrete due to the bond is greatest near the bar, thus forming a cone of deformation about the bar, and smaller deformations or strains occur between consecutive bars in the plane of the reinforcing steel than take place at the surfaces of the bars themselves. This condition of deformation causes some of the energy of internal work to be performed at an angle to the axis of the bar, or laterally, thus differentiating the reinforced concrete beam from the beam of homogeneous material in relation to the storage of internal energy.

It cannot be maintained consistently that burying the steel in the concrete changes its molecular properties or its modulus of elasticity, because, when we dig the steel out after it has been embedded in the concrete, we find that it is just the same kind of steel as it was before it was embedded, and possesses the same properties; and any conclusion to the contrary is unwarranted. It cannot be maintained that the joint action of the combination is equivalent to the resistance furnished by a plain concrete beam without any steel whatever plus the resistance of the tensile stress in the steel multiplied by its lever arm, because experiments in which the concrete is divided or separated at mid-span of the beam by pieces of tin so that no tensile stress can be transmitted in the tensile zone from one-half of the beam to the other by the concrete, show to a large extent the same phenomena of apparent but not true increase in the modulus of elasticity of the steel when measuring its deformation at mid-span and right across the line where it is dissociated from the concrete.

The subject of horizontal shears in vertical planes is one that is little understood by the Engineering Profession, or by theoretical writers on the subject of reinforced concrete. Lack of familiarity with this kind of action has led to an erroneous report by the Joint Committee as to the strength of reinforced flat slab and column construction, and to the unwarranted assumption in the report that the tensile strength of plain concrete in the slab can account for the enormous increase in strength which it exhibits over beam action in the column-supported flat slab. Less assumption and more experimental work and a careful discussion thereof would have resulted on the part of the Joint Committee in a far different report, and one which would not be questioned, as this one will be by all who have had any practical experience in the building of structures of this type.

Mr. Turner.

This phenomenon of reinforced concrete is not so simple and apparent as it may now seem, even after the explanation has been discovered. It required nearly 3 years of hard study and investigation before this satisfactory explanation was obtained. This question was discussed in a paper presented to the Boston Society of Civil Engineers, in September, 1914, in which the phenomenon was compared in action to a thrust applied at the end of the beam. The law of conservation of energy applied to the problem would prevent a consideration of the tensile stress in concrete multiplied by its lever arm as balancing that portion of the applied moment not carried by the steel, because of the lateral action introduced in the cone-shaped deformation through the operation of bond stress about the bar toward the end. These internal shears seem to be somewhat equivalent in their action to the effect of a thrust applied at the end of a beam, and not greatly interfered with by cutting the concrete through the tensile zone at mid-span.

Although the operation of horizontal shears in vertical planes is not to be depended on in a beam, because the utility of this action is destroyed by the cracking of the concrete toward the end of the beam, the conditions are different in the continuous flat slab floor, where the tendency to rotate is predominantly about a vertical instead of a horizontal axis, and, for that reason, is dependable.

A further point of interest lies in the fact that although every pound of pull in the steel is brought on it by bond stress, the work of internal deformation in tension is largely stored within the steel, a conclusion which was far from being apparent in the first consideration of the subject. Static balance merely requires equal opposing forces, with no restriction as to the amount of work performed directly by these forces, so that although the steel may store very largely the internal work of tensile deformation, horizontal shears in vertical planes may have an equal effect in reducing deflection without performing the same relative amount of work, since they operate in a radically different manner and through a much shorter range of action.

The writer presented some tests at a meeting of the Concrete Institute relative to this subject several years ago, but has not been able to supplement them by further experimental work, as he had hoped to do by this time.

Mr. Scott deserves the thanks of the Profession for clearly bringing out the incongruities of the Joint Committee's theory which he notes and which the writer has endeavored to explain. Figs. 8 and 9 show, in an exaggerated manner, the condition discussed.

Mr. Turner.

The impossibility of assigning any dependable value to the tensile strength of the concrete in a floor may be made apparent by the following experiment conducted by the writer. A slab, about 24 ft. square and $5\frac{1}{2}$ in. thick, supported on the edges on beams and piers and by a center pier, was loaded as shown in Fig. 10. It carried this load, consisting of concrete barrels filled with water, without signs of failure when first applied, but a drop of temperature about

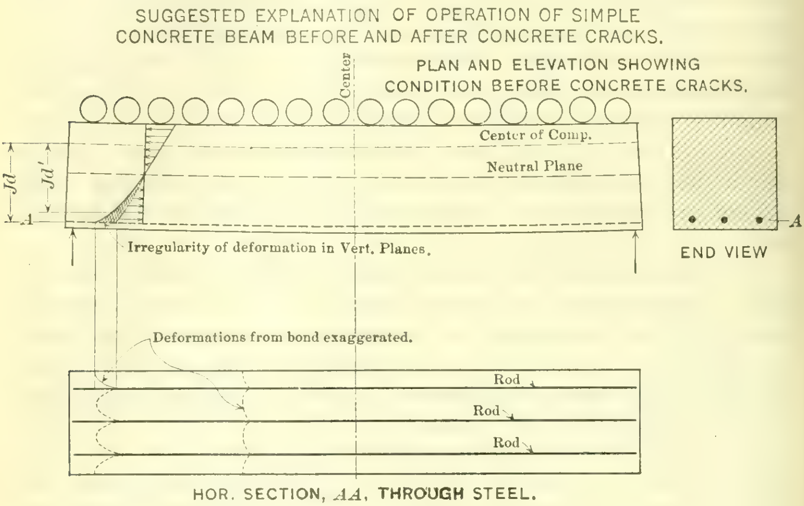

FIG. 8.

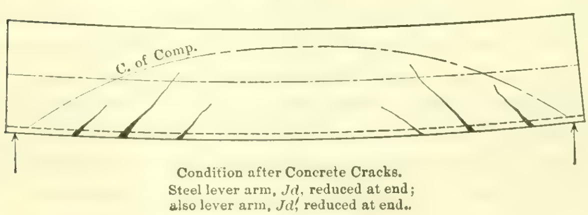

FIG. 9.

25° caused the failure shown in Fig. 11. If a change or drop in temperature of only 25° is sufficient to overcome the tensile strength of concrete, even when assisted by a little wire netting, as was the case here, the error of relying on such resistance as furnishing any dependable load-carrying capacity in the average building, which is subjected to greater reduction of temperature than this test slab was, is apparent, and needs no further comment.

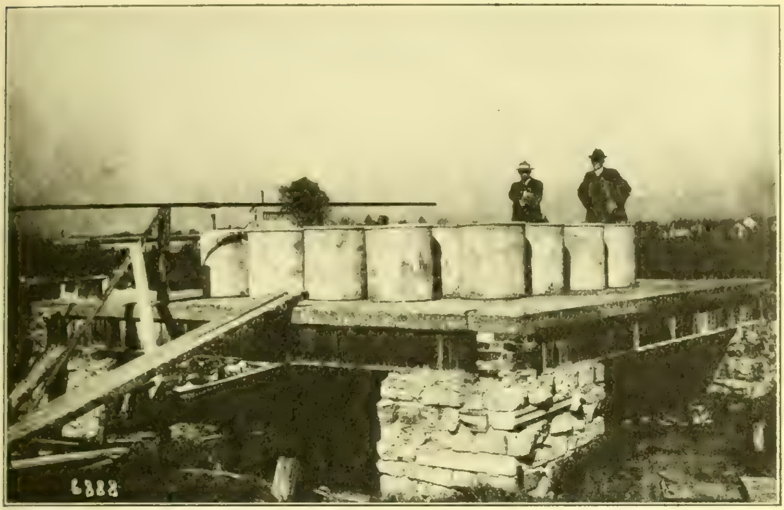

FIG. 10.—SLAB CARRYING LOAD WITHOUT DISTRESS, VERY LARGELY BY VIRTUE OF THE TENSILE STRENGTH OF THE CONCRETE.

FIG. 11.—FAILURE OF SLAB SHOWN IN FIG. 10.

F. E. TURNEAURE,* M. AM. SOC. C. E. (by letter).†—Professor Eddy's discussion‡ of the report of the Joint Committee raises certain fundamental questions which should be answered. It appears to the writer that his analysis at certain points is absolutely incorrect, and should not go unchallenged. In the first place, on page 887,‡ he criticizes the Committee for applying the theory of statics to the flat slab, and for assuming flat slabs "to be statically determinate structures." The Committee makes no such assumption, and neither does Mr. Nichols. The Committee is well aware that such structures, like continuous beams, are not statically determinate, but it does not follow that the law of statics cannot be applied to them. To be statically indeterminate simply means that the unknown quantities in the problem are too numerous to be determined by the equations of statics alone. Certain total shears and moments, however, can be determined by statics, although the exact distribution of these shears and moments cannot be thus determined. This is the position of the Committee. Mr. Turneaure.

It is interesting to note that, after criticizing the Committee for applying the laws of statics to this case, Professor Eddy, on page 888,‡ proceeds to do the same thing, where, in Paragraph 3, he applies the laws of "equilibrium" to determine total shears—a perfectly correct procedure, and exactly the same sort of thing as that done by the Committee. The same process can be used to determine total bending moments along either axis, and it is quite unnecessary to make use of any involved mathematical process to do this. Although the total bending moments can be determined in this way, their distribution along the slab cannot, and, in this degree, the problem is statically indeterminate.

Professor Eddy finds that the total shear around the edges of a square slab is equal to the total load—a result with which the writer fully agrees—but the results obtained by his analysis for total bending moments are quite erroneous. For a square panel, he finds that the numerical sum of the positive and negative bending moments along the center line, and along one edge, is $\frac{1}{16} W l$, whereas the correct value is $\frac{1}{8} W l$ (neglecting the effect of size of column in reducing slightly the bending moment). As this question is a very fundamental one, and as the conditions of the problem may easily result in confusion in applying the ordinary laws of statics, it will not be out of place to discuss this matter on a rather elementary basis.

* Madison, Wis.

† Received by the Secretary, July 6th, 1917.

‡ *Proceedings*, Am. Soc. C. E., May, 1917.

Mr. Turneaure.

Consider a square panel, Fig. 12, loaded uniformly with a total load, W. Assume that the columns are equivalent to square columns, in accordance with Professor Eddy's conclusions on page 889.* For simplicity, consider that the columns are small as compared to the size of the panel, so that the size of the columns need not enter into the expression for bending moments. The total shear at the columns will be W, and the shear on each of the surfaces, AB, BC, etc., will be $\frac{1}{8}W$. There will be no shear on the lines, AG, CD, etc. These statements agree with Professor Eddy's conclusions.

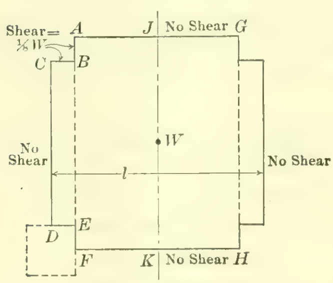

FIG. 12.

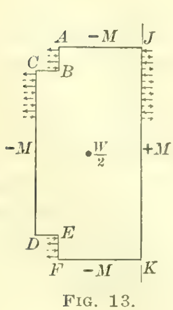

FIG. 13.

Now proceed to the question of moments: Consider the half-panel to the left of the center line, JK, Fig. 13. There will be bending moments at all points of the periphery, these moments acting in a direction at right angles to JK along the surface, JK, and also along the surfaces, AB, CD, and EF. Moments along the surfaces, AJ, CB, DE, and FJ, will act in a direction parallel to the axis, JK. The load on the half-slab is $\frac{1}{2}W$ and the shears are $\frac{1}{8}W$ on each of the surfaces, AB, BC, etc. Now, taking moments about the axis, JK, the external moment is that of the couple formed by the load, $\frac{1}{2}W$, and the four shears of $\frac{1}{8}W$ each. This moment is practically equal to $\frac{1}{8}Wl$. For equilibrium, the sum of the internal moments must also equal $\frac{1}{8}Wl$, and the internal moments in question are the positive moments along JK and the negative moments along AB, CD, and EF. That is to say, the numerical sum of the negative moments along a line through the columns and the positive moment along the center is equal to $\frac{1}{8}Wl$, as given by the Committee.

This method of analysis is a simple, direct application of the laws of equilibrium, quite as definite as the application of these laws in determining shears. No amount of mathematics will enable the conclusion to be reached that the internal moments acting perpendicular

* *Proceedings*, Am. Soc. C. E., May, 1917.

to the surfaces, AJ, CB, etc., can help balance the bending moments about an axis, JK, at right angles thereto. Mr. Turneaure.

In contrast with this result of $\frac{1}{8} W l$, Professor Eddy arrives at $\frac{1}{16} W l$ by a process of analysis in which he omits the effect of the shears on the surfaces, CB and DE, considering that these shears affect only the bending moments parallel to JK. As a matter of fact, the shears on CB and DE are a little more effective in producing bending moments about the axis, JK, than those on AB and CF, on account of the increased lever arm. The fallacy of Professor Eddy's position may be brought out clearly by noting what happens if we consider the portion of a slab limited by the lines AF and GH, Fig. 12. Neglecting the small load applied directly to the strip, $BCDE$, and the similar strip on the right, the total shears on the lines, AF and GH, must be equal to the load, W, by the same process of reasoning as that heretofore used. Then, if we consider one-half of this slab and take moments at right angles to the axis, JK, as before, the external moment will be plainly $\frac{1}{8} W l$, which must be equal to the total internal moments along the lines, AF and JK.

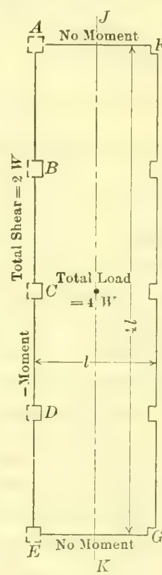

FIG. 14.

The inevitable conclusion, therefore, is that the numerical sum of the negative and positive moments along the edge and center line of a square panel is approximately $\frac{1}{8} W l$, and not $\frac{1}{16} W l$.

The matter may be clarified still further, perhaps, by considering an entire row of panels across a building. Suppose, for example, that the floor is four panels wide. In Fig. 14 is shown a row of four such panels, AF and EG being the exposed edges of the floor; AE and FG are sections through the floor. The columns are at A, B, C, D, etc. There are no bending moments on the edges, AF and EG. The total load is 4 W, and the sum of the shears at Columns A, B, C, D, and E is 2 W. The shears at the several columns are no longer exactly equal, on account of the discontinuity at A and E, but the sum is the only quantity needed. Now, take a section through JK, and consider moments on the left. The external moment is $2 W \times \frac{1}{4} l = \frac{1}{2} W l$. This is equal to the sum of the internal moments on the lines, JK and AE, acting perpendicularly to JK.

Mr. Turneaure. The average moment per panel will be one-fourth of this, or $\frac{1}{8} W l$, as previously found. This result seems very clear, as, in this case, there are no internal moments along AF and EG to confuse the issue.

This analysis leads to the conclusion that the sum of the moments on JK and AE is precisely the same $(2Wl)$, no matter what the number and spacing of the columns along AE and FG may be, as long as the arrangement is the same on each side, so that there is no shear on the line, JK. If the supports consist of continuous walls along the lines, AE and FG, the same result will be reached.

This leads to the general conclusion that, with symmetrical conditions with respect to the axis, JK, the sum of the internal moments along AE and JK is precisely the same whether the supports be walls, or columns spaced in any manner, a result which is not incompatible with the fact that the structure is statically indeterminate. Professor Eddy seems to agree that the bending moment for wall supports in the case considered would be $\frac{1}{2} W l$, but maintains, in effect, that if we take away most of the wall, leaving only small columns, the moment will be immediately reduced to $\frac{1}{4} W l$. As a result of this, one-half of the reinforcing rods may be taken out and placed in the other direction and the slab be made considerably thinner. This is an astonishing result, and obviously incorrect. The interior moments in the slab acting parallel to the axis, JK, cannot help out the moments transverse thereto. They have their own duty to perform, namely, to carry the load from column to column in the longitudinal direction, a result brought about by changing walls into columns.

Mr. Talbot. A. N. TALBOT,* M. AM. SOC. C. E. (by letter).†—Professor Eddy's discussion‡ contains so many fallacies and so many violations of accepted principles of mechanics that it is hardly believable that it is presented seriously by one who is familiar with the analysis of the action of structural parts. Since reasoning presented in mathematical form and coming from one of high standing may appear plausible, the erroneous statements and conclusions may mislead, and a more explicit answer is needed than will seem necessary to some readers.

Professor Eddy puts forward the astonishing statement that "any assumption of the validity and applicability of statical analysis to continuous flat slabs is incorrect, and leads to erroneous results, just as much as in the case of continuous beams or of any other indeterminate structures." That statical principles are valid for all structures is generally accepted, and it is difficult to believe that Professor

* Urbana, Ill.

† Received by the Secretary, July 23d, 1917.

‡ *Proceedings*, Am. Soc. C. E., May, 1917, p. 876.

Mr. Talbot.

Eddy intends to deny this generally accepted truth. Static analysis does not reveal all the conditions of indeterminate structures. For example, it cannot give the distribution of the intensity of the resisting moment along sections of the flat slab. Nevertheless, the indeterminate structure must be amenable to the statical laws of forces and moments, even though Professor Eddy seeks to exempt the flat slab from such laws.

On page 893* he gives a mathematical derivation of what he terms the total applied bending moment. In this derivation he uses the accepted relation between vertical shear and rate of change of bending moment, $dM = S_x dx$. In determining S_x, he adds the total vertical shear at a section, AB (Fig. 15), and the total vertical shear at a section, CD, and equates this sum to $\frac{Wx}{L}$ — the latter being the load between one of these sections and a middle section of the panel. It is evident that $\frac{Wx}{L}$ is the value of the total vertical shear on either one of the two sections, and not that of the sum of the total vertical shears on the two sections. He does not explain why he unites the shear of these two "free bodies" in the expression and yet considers the external load on only one. By this violent assumption, at one stroke the bending moment found in his analysis is cut in two. If the analysis was in any way defensible, the resulting moment coefficients would produce a wonderful saving in the dimensions and costs of structures. As the analysis involves a gross violation of the principles of mechanics, one must brand it as fallacious. That the sum of the numerical value of the positive moment at a mid-section and the numerical value of the negative moment at an edge-section is $\frac{1}{8} Wl$, as a limiting value for very small capitals, and not $\frac{1}{16} Wl$, as given on page 894*, rests on as firm a basis of logic as do any of the accepted expressions for bending moment in beams, girders, and other structures.

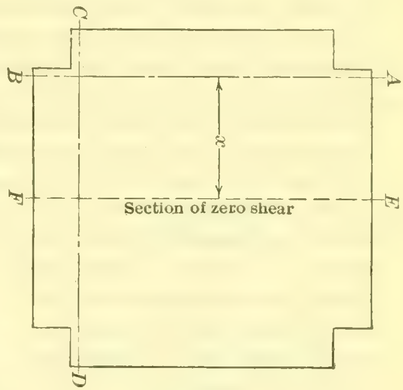

FIG. 15.

On page 890,* Professor Eddy states that the total vertical shear in sections parallel to the sides of the panels of flat slabs is 50% less than in a wide beam structure which rests on parallel walls, and

* *Proceedings*, Am. Soc. C. E., May, 1917.

Mr. Talbot.

that the total vertical shear in a section parallel to one of the sides of a panel is only that which arises from a uniform load of $\frac{1}{2} W$. This means that the vertical shear along a line, AB (Fig. 16), close to the supports, is only $\frac{1}{4} W$, or less. It is plain that such a statement violates the principles of equilibrium of forces acting on the portion of the slab, $ABFE$, principles which are just as applicable with a continuous slab having sections of zero shear along boundaries, except at the supporting columns, as they would be for an isolated panel. It seems too elementary to require pointing out that, for a uniform load over the whole panel, the total vertical shear on the section, AB, is once the load between AB and CD and not one-half of that amount. The presentation made by Professor Eddy assumes that a series of vertical forces along a side of a square which lies in a direction parallel to one side of the panel (shear along one edge of a square column) does not enter into the determination of equilibrium for sections at right angles to this side. This method of cutting the shear in two is not even ingenious, and it seems hardly necessary in a discussion among engineers to add that it has no logical basis. Of course, the acceptance of Professor Eddy's value of the shear would result in dividing the moment by two.

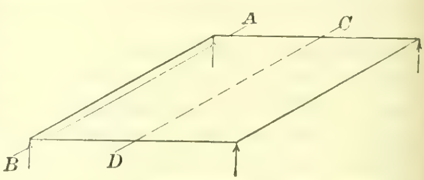

FIG. 16.

Attention may be called to the different view put forth by Professor Eddy in a paper* before the American Concrete Institute in 1916. In this paper he derives and accepts the value $\frac{1}{8} WL$ as the applied bending moment each way in a panel (the limiting value for very small capitals), instead of $\frac{1}{16} WL$, as given in his discussion. Further on in this paper, starting with $\frac{1}{8} WL$, he reduces the amount of moment necessarily provided for by a series of steps; (a) he divides it by two, because for two sets of reinforcing bars at right angles to each other, embedded in a matrix of concrete, the effect is the same as if the two sets of bars were a sheet of metal of the same weight, and hence a set of bars will have the same resisting effect laterally as longitudinally; (b) he divides it by two again, because as the bars do not slip in the concrete, the concrete will do as much work in a given direction as the steel; and (c) he states that tests appear to show that

* "A Further Discussion of the Steel Stresses in Flat Slab Floors," *Proceedings*, Am. Concrete Inst., Vol. XII.

the effect of Poisson's ratio reduces the stresses in the reinforcing bars of the slab to three-fourths of what they otherwise would be. The combined effect is to divide the moment by $5\frac{1}{3}$, and he gives $\frac{3}{128}\,WL$ as a final value for the sum of the positive and negative resisting moments, on which to base the design of the reinforcing bars, instead of $\frac{1}{8}\,WL$ before referred to. If these considerations were added to the effect of halved shear given in his discussion already referred to, the result ($\frac{3}{32}$ of that given by static analysis) would produce moment coefficients which would effect remarkable economies in structural design. Mr. Talbot.

Professor Eddy favors bending all reinforcing bars downward at a point fixed at the same distance from the panel edge. Although the position of the point of dip may be expected to affect that of the point of inflection somewhat, the differences produced for ordinary conditions will be slight, especially as the stiffness of the concrete in compression and tension has considerable influence. The recommendation in the report provides for such changes in the position of the point of inflection as may result from a change in the distribution of the load on the floor or from its concentration, and adds to the diagonal tension resistance of the slab, as well as avoids a possible plane of cleavage.

The actual distribution of the intensities of the resisting moments along a section of the panel will be affected by variations in resisting stiffness in different parts of the panel, but, as the concrete of the slab in its compressive resistance, and to a certain extent in parts of the slab in its tensile resistance, is a great factor in making up the stiffness of the slab, changes in the location or distribution of the reinforcing bars over a panel-wide section will not affect the distribution of intensities of resisting moment to any such extent as is claimed by Professor Eddy. The report allows a considerable leeway in the manner in which the reinforcing bars may be distributed over a section. It may be added that experiments have not borne out the claim for added strength in saddle-shaped portions.

Professor Eddy objects to the recommendations made for general thickness of flat slabs on the ground that it has been indubitably established that the ratio of steel reinforcement to concrete may be made greater in beams of relatively large depth over that permitted in beams of relatively small depth. The attempt, on page 878,* to establish this statement by the relation between the sharpness of the curvature and the horizontal shearing distortions in the concrete at points distant from the sections where maximum moments occur, is absurd. If true, the criticism would apply also to the principles of

* *Proceedings*, Am. Soc. C. E., May, 1917.

Mr. Talbot. design of all beams and girders. Besides, the relation of thickness to length existing in the ordinary flat slab is that of relatively long beams. In establishing values for the formula for slab thickness, Professor Eddy starts with two assumed examples of thickness of slab for definite panel lengths, considered to be known from experience, and, later (page 883*), he naïvely suggests that it will be sufficient to assume that l (given as the total panel length in the derivation of formulas and Table 2) is the effective span (a dimension shorter than the panel length) and may be so regarded in using Table 2. Professor Eddy seems to have overlooked the fact, which a little study will show, that the formulas given in the report for thickness of slabs with and without dropped panels are devised to produce working stresses in both reinforcing bars and concrete, which are in close agreement with the values of working stresses ordinarily used in design when the moments and their distribution are taken as recommended in the report.

On page 901,* Professor Eddy first uses $L - C$, and then adopts $L - \frac{2}{3} C$, as the span of the panel in determining the effect of the size of capitals. The expression for moment given in the report corresponds closely to the moment of the couple formed by the resultant of the load on the half panel and the resultant of the shears at the column supports. A more exact expression is found in the paper by John R. Nichols, Assoc. M. Am. Soc. C. E., on "Statical Limitations Upon the Steel Requirement in Reinforced Concrete Flat Slab Floors."† That given in the report is a close approximation to the more exact expression.

Any information or analysis which will furnish grounds for reducing coefficients and dimensions of engineering structures should be welcomed. To be acceptable, such grounds must stand the tests of the fundamental principles of mechanics and of the known action of structures. Doubtless, the future will see many developments in concrete and reinforced concrete. With increased knowledge and experience in construction, in the choice of materials, and in the methods of work, and with the greater confidence which may develop with continued use, it is not improbable that, in the future, it will be found proper to increase allowable working stresses to more than those now recommended, and even to encroach on the margin between allowable loading and the critical load which a structure will stand—the so-called factor of safety. Practice will change, and recommendations must be revised from time to time, but changes should not be based on misapplication of basic principles.

* *Proceedings*, Am. Soc. C. E., May, 1917.

† *Transactions*, Am. Soc. C. E., Vol. LXXVII, p. 1673.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

DISCUSSION ON REPORT OF THE SPECIAL COMMITTEE TO FORMULATE PRINCIPLES AND METHODS FOR THE VALUATION OF RAILROAD PROPERTY AND OTHER PUBLIC UTILITIES*

By Messrs. W. R. McCann and C. E. Grunsky.

W. R. McCann,† Assoc. M. Am. Soc. C. E. (by letter).‡—It is readily to be conceded that the Committee has produced a very creditable and momentous work. The task of developing any sort of unanimity from the chaos which has heretofore existed in matters pertaining to the valuations of railway and other public utility properties is an estimable performance that is appreciated best by those who are daily engaged in valuation work. The Engineering Profession as a whole, and particularly those members thereof who have to do with appraisals of public utilities, owe a sincere debt of gratitude to the authors who, by exhaustive research and by many conferences, have succeeded in producing a report which purports to represent and reflect the Committee's unanimous, composite idea of valuation theories. Such a report will carry great weight indeed, in the future, whenever it is quoted and referred to as an authority; and its two hundred pages will be fingered many times by youthful engineers, apprenticing in appraisal and valuation work. The extended scope of the report, and the important utilization of its contents, should render engineers quite cautious in concurring indiscriminately with all the statements and theories therein, even though advanced unanimously by the Committee. Mr. McCann.

It is not to be expected that the Committee could produce a report which would be generally approved by the Society's membership. It

* Continued from April, 1917, *Proceedings*.

† Washington, D. C.

‡ Received by the Secretary, June 8th, 1917.

Mr. McCann. is much more probable, judging from controversies which have arisen over the subject matter in the report, that every statement therein has its opponent. The question is, "Does the Committee's report on valuation of public utilities reasonably represent the status of the art to-day?" The writer believes that the question must be answered in the negative, particularly from the point of view of the regulatory bodies and the judiciary, which are charged by statute with establishing equities between utility companies and their consumers.

In a great many respects, the report is quite favorable to the positions heretofore taken by corporations. A careful study of it makes manifest the conclusion that hardly a scrap of a claim which will result in enhancing the value of utility property is neglected, unless it is obvious, after many a try-out before commissions and Courts, that the claim manifestly is to be disallowed. There is great danger in following the report implicitly; young appraisers and students, particularly, may be led astray, with the result that they may recommend (often advocate) absurd conclusions derived from adhering too strictly, without the exercise of mature judgment, to the Committee's leanings.

To point out all the places wherein there is much more to be said in behalf of the public's side than has been stated by the Committee would consume as much space as the report itself, and even more. Such extensive purpose is not the writer's intent in this limited discussion; he will confine his remarks to one important subject, that of "Original Cost to Date"—a matter which has been given but five pages, as contrasted with about a hundred pages (nearly one-half of the entire report) given to the antithetical theory, the "Cost of Reproduction."

It is to be frankly conceded that much of the language of the Courts to date has been favorable to the reproduction theory of valuing utility property; and it is also opportune to remark that a considerable portion of the judicial language has been misquoted, misrepresented, and misconstrued by partisan advocates. It is not to be overlooked, moreover, that the strength of original cost is to be found in commission decisions; and this is especially significant in that those who devote their entire time to the solution of valuation, rate-making, capitalization, and other problems affecting public utilities have marked leanings toward the equities inherent in the original-cost method of valuation. In the writer's opinion, the day is not far distant when the Courts, on review of a record which contains a commission decision embracing a valuation based on a properly compiled original-cost appraisal, will comment much more favorably on the original-cost method than they have on divers reproduction theories.

Heretofore considerable discredit has been given the original-cost method, due to confusion which has existed as to what is meant thereby. The capital account carried on a utility's books has been taken to mean the original cost of the property; likewise, it is argued that the purchase-and-sale price in a trade consummated years after the original construction represents the original cost of the property; also, the original sum invested in the nucleus of a property has been assumed to be representative of the original cost. These are not correct definitions. A proper original-cost appraisal is as capable of precise definition as is any reproduction theory. Before proceeding further with this discussion, it is well to bring out what constitutes a sound original-cost appraisal. Mr. McCann.

The California Railroad Commission,* submits the following definition of the term "original cost":

"The term 'original cost' means the actual expenditures chargeable to capital account, in accordance with the Interstate Commerce Commission's or this Commission's classifications, in cash or its equivalent in terms of cash, by the public utility for its property in the State of California, as of the date of the valuation."

Mr. Hammond V. Hayes,† a consulting engineer of Boston, defines actual original cost, thus:

"The actual original cost is the sum of money which was expended by the undertaking for the property now in use for the benefit of the public. It is not what the original property cost but rather what the present property cost. The expression 'original cost' is liable to convey a false impression. What is required in a valuation is the 'actual cost' of the property now in use. The term 'original cost' has been used so generally in decisions of courts and commissions, however, that it cannot be now eliminated. * * *

"The actual original cost should not be considered to be the cost of the first unit of plant used in a particular place or for a particular purpose. Items of perishable property, which are no longer in use or useful, cannot be considered as a portion of the property to be included in a valuation for the purpose of determining the fair value for rates. Such items of property have passed out of existence and their cost should have been removed from the books of the company as a portion of the value of its assets. If the business of the undertaking had been conducted properly, reserves for renewals would have been made. These reserves are obtained from users as a portion of the charges paid for the service. Manifestly it is unfair to the users for the company to demand from them rates sufficiently high to create a fund for the replacement of obsolete items and then include the cost of such obsolete items in a new value upon which new rates should be based."

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* Annual Report for the fiscal period from July 1st, 1915, to June 30th, 1916, p. 92.

† "Public Utilities: Their Present Fair Value and Return," pp. 123-124.

Mr. McCann.

Mr. Robert H. Whitten,* of Brooklyn, likewise defines actual original cost:

"Strictly speaking, actual cost means cost of original construction plus cost of additions and betterments. It excludes all expenditures for renewals and replacements including supersession due to obsolescence or inadequacy. It includes only construction, additions and betterments that are a proper capital charge under approved accounting principles. This conception of actual cost, however, is one that has in the past been very imperfectly comprehended. Correct accounting principles are of comparatively recent acceptance and application. The references made by courts to actual cost or original cost plus improvements show that in most cases they have loosely interpreted the term to include many things that are not properly a part of the actual cost of the present property. In certain decisions it is apparently assumed that actual cost or original cost includes discount on securities issued, exorbitant profits to promoters, cost of replacing worn-out or superseded property, dividends paid out of capital, money sunk in unsuccessful experiments. That is, the term is considered as an equivalent to book value inflated by financial manipulation or loose accounting. Considered in this light, it is little wonder that 'original cost' has been discredited as a standard of valuation.

"Actual cost properly considered is the most natural and in many respects the fairest single basis for the determination of fair value for rate purposes. A fundamental principle of public service regulation is that as the public service corporation devotes its property to a public use it may consequently be required to render the service at reasonable rates of charge. Rates of charge to be reasonable may not be in excess of the fair value of the service and may not be higher than necessary to produce a fair return on the property devoted to a public use. The measure of the property devoted to a public use is undoubtedly, in the first instance, at least, the money that the company has actually and necessarily invested, *i. e.*, the actual cost."

Mr. Halford Erickson, ex-Chairman of the Wisconsin Railroad Commission, in a paper entitled "Depreciation and Its Relation to Fair Value", read before The Utilities Bureau in November, 1915, comments similarly on the definition and application of an original-cost appraisal, as follows:†

"By original cost in this connection it seems to me should be understood the cost at which the existing property used by public utilities in rendering service was acquired. By cost of reproduction is meant the cost of reproducing the existing property under prevailing conditions. The original cost of the existing property should be shown by the books and records of the utilities provided these have been properly kept and are still in existence. When the books have not been so kept or are not available, the original cost as thus outlined may be determined very much in the same manner as that in which the cost of reproduction is found. * * *

* "Valuation of Public Service Corporations," pp. 82–83.

† *The Utilities Magazine*, 1-3-113.

"When the original cost of the existing property is desired it can be computed upon the same inventory as that used in determining the cost of reproduction and upon prices which cover the period when the property involved was put into the plant. Such price lists may be had partly from the records of the plant and partly from other sources. In this way the original cost of the existing property can be had with even greater accuracy than the cost of reproduction." Mr. McCann.

The following statement* of the Committee follows the identical line of reasoning, to wit:

"As original cost to date, with comparatively few exceptions, is not the book cost of the property but the cost of the existing items, it will be necessary as a rule to make a schedule of the various existing property items, in the same way that one would be made for determining the cost of reproduction; then reference would have to be made to the accounts, to ascertain the unit costs of the items."

The term "Original Cost" is used by the Engineering Department of the State Public Utilities Commission of Illinois to mean the actual expenditures, in cash (or its equivalent in terms of cash), made by a public utility for used and useful property which is properly chargeable to capital account and is embraced in an actual inventory; or, in the absence of records and books of accounts showing the actual expenditures, the estimated cost of the property as of the sundry dates of the installations of the various items of property is to be considered the original cost thereof.

Bearing in mind the aforesaid definitions of what constitutes a reasonable original-cost appraisal, particularly the definition set up in the preceding paragraph, it is well to refer to certain passages of the report, wherein the strict reproduction theory is modified and violated in principle, and recourse is taken in effect to the original-cost method. Perusal of the report, moreover, would seem to indicate that the principles of a pure reproduction theory are adhered to whenever it is to the general advantage of the property under consideration, and that historical reproduction cost (another name for an enhanced original cost) is resorted to whenever the strict reproduction theory fails to give the "top-notch" figure, or fails to arrive at the known original cost.

The first specific example wherein the Committee recommends a departure from the strict reproduction theory, in favor of the historically modified reproduction theory, is found on pages 1761 to 1763 (inclusive) of the report.† Here there are cited instances of three different dams and reservoirs which were built under entirely different circumstances respecting contingent and auxiliary costs. The Wachusett Reservoir required the obliteration of several buildings,

* *Proceedings*, Am. Soc. C. E., for December, 1916, p. 1753.

† *Proceedings*, Am. Soc. C. E., for December, 1916.

Mr. McCann. highways, railroads, and other property formerly on the present reservoir site. The Kensico Dam, built within the limits of an existing reservoir, required the construction of two temporary reservoirs to maintain a water supply. The Au Sable Dams were constructed under simple circumstances, unaccompanied by exceptional costs of damages, removals, or temporary works. The Committee asks if it is fair, for the reasons stated, "to assume like conditions to govern the estimate of reproduction cost to be used as a basis of 'fair value'".

It certainly is apparent that these three dams and reservoirs are not to be appraised under "like conditions." The strict reproduction theory, however, calls for an appraisal of the existing property as it is to-day. Such a premise is satisfactory, say, for the Au Sable Reservoir, which did not encounter any unusual auxiliary costs during its construction. The strict reproduction theory, however, will not produce sufficient value for the Kensico and the Wachusett Dams; therefore the Committee, following in the footsteps of experts who have forsaken and modified the strict reproduction theory, "is of the opinion that all such items of cost, due to damages, destruction of property, purchase of rights, and temporary works, as cited in these illustrations, are clearly proper items to be included in the reproduction estimate when capable of historic proof", and this despite the fact that original-cost appraisals of these dams would reveal, in a very few items, the exact total sums expended in each of the three works.

It is not amiss at this point to contrast the proposed historical reproduction method, advocated by the Committee, with the original-cost method defined hereinbefore. In the Wachusett Dam, for instance, the appraiser must make a complete survey of the dam and reservoir as it is to-day; then he must apply unit costs of labor and material at prevailing prices to-day. Thus, the appraiser derives the reproduction estimate of the existing structure, whereupon he starts to conjecture what may have happened during the years of construction. Imagination is brought into full play in picturing the day-by-day and month-by-month changes in the scenery before him—once an industrious valley, teeming with activity, possessing thriving railroads, and incrusted with dust from many roads, now simply a peaceful lake. Many things might have happened during this transformation, and therefore the appraiser is restrained in his imaginative process to what actually did happen. The records of the work must be consulted, inasmuch as fascinating conjectures must be proved before commissions and Courts. Should the appraiser wander very far from the recorded facts and the actual cost thereof, he is in difficulties, if a keen counsel cross-examines him thoroughly as to the details underlying his theory. How much simplier it is to set forth the book-recorded costs and quantities of a few principal items, such as clearing, sanita-

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tion, excavation, temporary works, construction camp, masonry, earth fill, spillway, moving structures, moving highways, moving railways, etc. In arriving at fair values, commissions and Courts are not in the habit of disallowing sums which have been prudently and wisely expended in the construction of used and useful utility property; instead, their problem is to isolate the reasonable from the limitless conjectures of experts and from the flowery arguments of attorneys who seek to inflate the much-abused term "value."

The remarks of the last paragraph apply equally well to several other specific examples set forth by the Committee. The instances of the Pennsylvania and New York Central Terminals, on page 1763, are very similar to the aforesaid dam examples. There is little doubt in the writer's mind that excellent cost data are available for the recently-constructed New York terminals. Why is it necessary to disregard the actual original cost of these terminals and turn to a conjectural reproduction thereof? Opponents to the reproduction theories will answer that the underlying reason lies in the possibility of claiming inequitable increments in the railway's capital account. Advocates of the reproduction theories will answer that the purpose is to arrive at present value; and then the deluge of arguments of what constitutes value soon creates mire sufficient to obscure the original issue.

The next example cited by the Committee is that of a certain highway crossing a railroad track. For the sake of brevity, the Committee's statement* relating thereto is reproduced, as follows:

"One illustration brought to the attention of the Committee is a case of a highway which originally crossed a steam railroad at a very acute angle, making a dangerous crossing to both railroad and public. At its own expense the railroad acquired land parallel with its own, changed the course of the road for several hundred feet and made a right-angled crossing. It was argued that, 'in case of reproduction, all other property remaining as at present', the highway would be in its new location and would not have to be reproduced. On this basis, many items which the owner of a railroad or other public utility was compelled to pay for might be cut out of an appraisal. But the highway would be in its original place were it not for the act of the railroad, and would have to be moved by any railroad, building presently, if the existing road had not been built; the evidence of the necessity, however, having been destroyed in the doing. The cost was a proper cost, capable of historic proof; and the Committee believes that it should be included as a cost in the estimate of reproduction."

If the writer understands this last illustration correctly, the cost of the old highway crossing (which had grown inadequate to meet modern traffic condition, and was dangerous), should be charged against the railway's accrued depreciation fund, created for the purpose of replac-

* *Proceedings,* Am. Soc. C. E., for December, 1916, p. 1764.

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ing equipment, structures, and other property as it wears out, grows obsolescent, and becomes inadequate. The actual cost, borne by the railway, of the new and re-aligned highway crossing should be placed in the capital account of the carrier; and such an item would appear in simple terms in an original-cost appraisal, without theorization of what may have happened during a process of reproduction.

It seems needless to dwell further on the several instances wherein the Committee advises departure from the strict reproduction theory and recommends the historical modification thereof. The writer regrets that the Committee did not see fit, in each of the examples, to point out wherein the original-cost method would establish more convincing equities before a tribunal in a valuation proceedings.

Before leaving the Committee's specific examples of application of a reproduction theory, attention is called to the stone bridge piers, instanced on page 1771 of the report, and to similar examples following. Certain abutments of a railroad bridge, built of stone masonry at a cost-to-reproduce of from \$12 to \$16 per cu. yd., in accordance with the road's practice for the previous 10 years, would now be built with concrete at a cost of from \$7 to \$11 per cu. yd. The Committee's conclusion, as to this and similar examples, is that the precise existing structure should be appraised at its reproduction cost, and that the only departures (page 1772) from such a rule are the cases where, due to obsolescence of existing machinery, it is absolutely absurd to pursue this method to its ultimate conclusion. Once again comment is forthcoming to the effect that the Committee's recommendations result in an adherence to the strict reproduction theory whenever the value to be derived therefrom enhances the original cost of an item of property; but, if the strict theory does not establish sufficiently high values, it would seem that the Committee's idea is to modify the theory. If a theory can be modified in one place, there should be no great objection to a reasonable modification at another place. Unfortunately, multitudinous modifications of the strict theory result in individual theories of each and every appraiser. The writer, in the instance of the masonry abutments referred to, would call attention to the equities of appraisal at the original cost of the structure, regardless of what similar stone masonry is costing to-day, and regardless of the present-day cost of better concrete abutments.

Possibly it will not be amiss to cite here a pertinent criticism made by the late John M. Eshleman, formerly Lieutenant Governor of California, and once President of the California Railroad Commission, in an address* delivered before a conference held under the auspices of The Utilities Bureau, at Philadelphia, in November, 1915:

"* * *. The inconsistency of adopting the historical method with reference to some elements and rejecting it with reference to some

* *The Utilities Magazine*, 1-10-11.

others need only be mentioned. Why it is that the committee of presidents of the railroads of the United States should concern themselves with hidden costs of some elements of the properties of the railroads and forget all about hidden or other costs of lands and properties that have appreciated, is beyond my comprehension. Mr. McCann.

"I really seriously wonder at the logic of those who urge that cost shall ever have anything to do with this question unless they admit it has everything to do with it, and I marvel at their regard for the simplicity of public authority when they urge that original cost shall always be used even in their reproduction theory when it will give them more than present cost to reproduce, and at the same time utterly repudiate costs of lands and similar properties when such costs are less than the market value now obtained. I do not for a moment contend that the work of the engineers in making inventories, and, if you please, in fixing unit prices is not very important. But holding, as I do, that where government is dealing with monopoly it of necessity must deprive such monopoly of the power of taking all that it can get from its patrons, I am driven to conclude that government must determine as a substitute what the agency ought to take, and in determining what the agency ought to take, the safest and most just guide is what the agency has sacrificed and what service it has performed for the public. * * *"

That the original-cost method possesses merit in valuation proceedings is shown by such facts as (1) a more or less general adoption by regulatory State commissions of original-cost as a perfectly legitimate line of inquiry to shed light on property values, (2) the recognition by Congress of original cost as one of the methods to be pursued in the valuation of the nation's railroads, (3) the National Association of Railway Commissioners' unanimous insistence that original cost be considered in the Interstate Commerce Commission's valuation of common carriers' properties, and (4) Court decisions supporting the position of the Supreme Court of the United States in Smyth *v.* Ames (cited by the Committee). No judicial authority, moreover, is to be cited to show that an estimate of the cost of reproducing the property (with or without deduction for accrued depreciation) is the sole and only guide to a reasonable and adequate valuation of a utility property for rate-making purposes. On the contrary, the inconsistencies of the reproduction method have been scored time and again. It is only recently that the Supreme Court of the United States, in the Des Moines Gas Case, repudiated the reproduction method when applied to what, in valuation work, is commonly termed "undisturbed paving."

In addition to those steadfast older commissions and their representatives, who have given due and just weight to the actual original-cost methods, it is noteworthy that many a recent author is outspoken in recommending the sacrifice of the investor as the measure of fair value, particularly fair value as a rate base. (Cf. Whitten and Hayes, *supra.*) A recent engineering contribution is "Public Utility Rates," by Mr.

Mr. McCann. Harry Barker, Associate Editor of *Engineering News,* wherein the author (at pages 37 and 38) states:

"Present tendency seems to be to give more and more weight to figures of investment and investors' sacrifice in determining rate-basis worth. With the general imposition of proper accounting systems figures of investments and sacrifice can be more completely secured than in the past so that a better basis for value by investment can be established. Where regulation has been longest established, there investment has greatest weight; it is not unreasonable to expect that in the course of time the entire country may accept the theory. * * *"

The most common argument against the use of the original-cost method of valuing public utility properties, aside from the difficulties alleged to be encountered in securing records of such costs, is that appreciations in value of property (if such exist) are denied to the owners thereof. The reproductionists, however, do not point to any rule of common equity which entitles them first to earn an adequate return on their investments and then to participate in the profits which accrue to appreciation; but they rely on certain judicial passages, which in themselves are sound, although subject to linguistic abuses when applied unadvisedly to valuations that are made bases for rate schedules. Under certain rulings of the Courts, it may be argued that, even though a utility may steal equipment without being apprehended, and may convert that equipment into used and useful property in the service of the public, the stolen equipment must receive due recognition in a valuation and rate-making proceeding. Whether or not this view will prevail ultimately, under a continuance of State regulation, is a somewhat debatable question at the present time. A simple case will serve to illustrate the fallacy of too great weight given indiscriminately to appreciation in utility property. Assume that an electric plant, in a State where the laws provide for State regulation, costs \$100 000, and assume, further, that the regulatory body of jurisdiction, after investigation at the very start, has fixed rates such as will yield full operating expenses plus 5% per annum (\$5 000) for accruing depreciation and 7% per annum (\$7 000) for a fair rate of return. At the end of 5 years, provided no change is made in the electric property, the utility will have accumulated \$25 000 (plus earnings) in a depreciation fund, and each year will have paid a full and adequate rate of return on the entire investment. During these 5 years, if perchance the prices of labor and materials advance so that the estimated reproduction-cost-new of the identical property at the end of the period, according to expert appraisers, would be \$110 000 (exclusive of betterments and additions), there will have been an unearned increment, over and above a fair rate of return, amounting to \$10 000 in the value of the property—equivalent to \$2 000 per year, or 2% annually on the original cost. In other words, a valuation thus made at the end of the said 5 years,

resulting in revised rates being fixed on an estimated reproduction theory, capitalizes an unearned increment which automatically results in rendering a 9% rate of return throughout the entire first 5-year period. What has the utility done or denied to itself in order to deserve this unearned increment? Is not the public entitled to participate in the appreciation of property, at least to the extent of not having the same capitalized against it, to be borne by the rate-payers of the future? Carrying the illustration still further, let it be assumed that the estimated reproduction value sinks to $90 000 at the end of 5 years; then the reverse is true, and the utility each year is deprived of just earnings equivalent to 2% of the cost of the property. Under such conditions, would it not be argued that this deprivation of earnings would constitute a confiscation of property? Mr. McCann

Such illustrations are indicative of the public's vital interest in what is termed "appreciation." It is to be borne in mind that "depreciation", as applied in valuation work, in no manner is the opposite of "appreciation." An original-cost valuation, if properly compiled, does not presume to inflict on a utility the losses occasioned by decreases in prices of material and labor. An original-cost valuation, however, does presume to reflect the conditions under which the bargain between a utility and its consumers was consummated. Under an original-cost valuation, the public sustains all losses due to the falling off of prices, although it participates in gains only to the extent of not having an unearned increment capitalized against it.

The reproduction theory has its place. It is undoubtedly one evidence of value, and an appraisal predicated on the strict reproduction theory should be made a part of every rate case; but the reproduction theory (or some modification thereof), under no circumstances, should be construed as the sole and only guide to value. Ultimate value for rate-making purposes is a discretionary exercise of judgment on the part of those governmental agencies which are empowered by legislative enactment to regulate public utilities, subject of course to judicial review should the administrative power be abused.

The Courts have not ruled that reproduction-cost-new (or less depreciation) must be the criterion by which to judge the present value of utility property for rate-making purposes; neither have the Courts stated that such reproduction-cost-new (or less depreciation) is not to be used, nor that original cost is not to be used. What the Courts have said is that all these evidences of value are elements to be considered with other factors in fixing on value for rate-making purposes. Whenever reproduction advocates turn their attention to bringing forth the advantages of the original-cost method as earnestly as they have expounded the reproduction theory, the disadvantages of the reproduction theory, which make it necessary to resort to original conditions and to

Mr. McCann. original-cost conceptions, will disappear miraculously in a strict adherence to the actual original cost method of valuation. A fair rate of return on the actual dollars prudently expended for utility property that is used in and useful to the rendering of public service are words that jingle with equity; and it is the writer's opinion that judicial sanction, supporting the position of commissions respecting original cost, is not long to be withheld.

Mr. Grunsky. C. E. GRUNSKY,* M. AM. SOC. C. E. (by letter).†—The fact that the profit to which the owner of a public utility is entitled may be expressed in terms other than as an increment of the rate of return applied to a rate-base, has not been brought out in the Committee's report, and yet this deserves discussion, as it smooths the way for determining reasonable rates without starting with "value." The purpose of this extension of the writer's discussion‡ is to make this fact clear. The Courts are compelled to determine whether or not the allowed earnings are confiscatory. Consequently, they have concerned themselves with value, regardless of how it may have been built up. The rate-regulating authority, on the other hand, should be and is free to use any proceeding which to it appears to be convenient and equitable in determining what the earnings and, therefore, the rates should be. This fact does not seem to have been recognized or accepted by many of the public service commissions, and they, therefore, following the lead of the Courts, are nearly all concerned with the value of public utility properties and find themselves forced, in some cases, as the writer has instanced in his preceding discussion, to read new meanings into the word "value." If every property to be valued was new, there would be comparatively little difficulty in estimating the amount that should be assumed to be reasonably and properly invested therein, but there is serious difficulty when a property is no longer new. The service rendered by, or the output of, a property which is mature and well-seasoned, is ordinarily better than, or at least as good as, the same service or output of a new plant. From the standpoint of the value of the service rendered, therefore, the rates determined for a new plant should be appropriate, too, for a plant the parts of which may show all manner of accrued depreciation. Except for the requirement apparently laid down by the Court that "value", interpreted to mean "present" or "depreciated" value, is to be taken into account, there is, in other words, no need at all of considering "accrued depreciation" (not to be confounded with "deferred maintenance") when rates are to be fixed. Conse-

* San Francisco, Cal.

† Received by the Secretary, June 5th, 1917.

‡ Previous discussion by Mr. Grunsky appeared in *Proceedings* for March, 1917, p. 503.

quently, as already stated, the principle of starting with a rate-base undiminished by depreciation is fundamentally sound. Mr. Grunsky.

When thus determined, however, something more than the rate-base is to be brought into consideration. The rate-base may express what the sacrifice was, which the owner had made and on which (to cover interest and expenses incident to securing money for the enterprise) he should receive an adequate interest return. In addition to an adequate interest return on such a rate-base, however, he may reasonably expect, and certainly is entitled to, compensation for management, and for business hazards; and he is entitled, also, to a participation in the prosperity of the community which he serves. In dealing with "value", it has become customary to include therein some allowance for "going value" and also to fix the rate of return on the resulting "fair value" higher than ordinary interest rates. A way has thus been found to permit the utility to earn more than interest on the investment. The business yields a profit. Unfortunately, it is difficult, if not impossible, to establish any fair or acceptable rule for determining the fair rate of return which ordinarily is to include the profit item. That this is difficult is due to the fact that one utility is of a character involving a large investment compared with the annual gross income, and another shows the reverse condition; a large amount of business is done on a small investment. Water-works may be suggested as of the first class, particularly when the areas owned for reservoir and water-shed purposes are extensive. Express companies suggest themselves as belonging in the second class. Their ownership of property and capital investment may be but nominal, although their annual receipts are large.

Why not give consideration, therefore, to the volume of business, to the annual gross income, as well as to a rate-base, when determining what the earnings should be? In other words, let the profit be brought into a fair relation to the volume of business, instead of attempting to predicate it on the rate-base, or that uncertain element, "value", which is, in part, created by the profit.

When an interest return is allowed to the utility, which will cover the cost of money borrowed for legitimate enterprises of the character of such utility, and this interest rate is applied to a rate-base estimated from legitimate proper investment undiminished by accrued depreciation, certain profit allowances about as noted subsequently will be found to be appropriate. It is assumed that, in determining the cost of borrowed money, due allowance will be made for discounts and commissions. The profit allowances, if made as suggested, would represent compensation in lieu of current appreciation and interest on the going value, or other elements of value not included in the rate-base. In the last analysis, however, the capitalization of

Mr. Grunsky.

this profit allowance would show that intangible values have actually been created and are demonstrable. The reversal of the usual procedure appears to be advisable. Instead of starting with the intangible values which are to be created by the earnings, let the profit be fixed, and the intangible values will take care of themselves. The suggested profit allowances are as follows:

When gross income is	$10 000	about	15%....	$1 500 per year.
" " " "	100 000	"	14.5%...	14 500 " "
" " " "	500 000	"	13.5%...	67 500 " "
" " " "	1 000 000	"	12.5%...	125 000 " "
" " " "	3 000 000	"	10%	300 000 " "
" " " "	5 000 000	"	9%	450 000 " "
" " " "	10 000 000	"	8%	800 000 " "
" " " "	20 000 000	"	7%	1 400 000 " "
" " " "	40 000 000	"	6%	2 400 000 " "
" " " "	100 000 000	"	5%	5 000 000 " "

The allowances here suggested as additions to the interest return on the rate-base are only tentative and remain subject to modification on further study. Furthermore, no such general suggestion as is here made could be expected to fit every case. These allowances, moreover, are intended to apply under ordinary conditions, and do not cover the larger return which the owner has a right to expect under the unusual condition when, by introducing a new invention or a less expensive process, which may be the direct result of his efficient management, he has reduced materially the cost of his output.

In such an event, the resulting benefit should go to both the owner and to the rate-payer, and the owner should be recompensed for discarded property, rendered useless perhaps by the new invention, but, necessarily, put at once into the class of property no longer in use. It would be proper in such cases, without imposing hardship on the rate-payer, to allow the rates to remain as they would have been without taking account of the reduced cost of production, at least long enough to amortize completely so much of the original plant as the new invention has rendered useless. After this amortization is accomplished, the rates will be subject to such adjustment as will bring the rate-payer into fair participation of the benefit which has resulted. If any policy less favorable to the owner was followed, he would find it to his advantage not to make any innovations which would require the abandonment of a part of his plant still in serviceable condition. He could not afford to assume new hazards resulting from the investment of more capital, unless he felt sure that his profits would be increased thereby.

MEMOIRS OF DECEASED MEMBERS.

NOTE.—Memoirs will be reproduced in the volumes of *Transactions*. Any information which will amplify the records as here printed, or correct any errors, should be forwarded to the Secretary prior to the final publication.

MENDES COHEN, Past-President, Am. Soc. C. E.*

DIED AUGUST 13TH, 1915.

Mendes Cohen was born in Baltimore, Md., on May 4th, 1831. His family, previously prominent in Europe, came to America before the Revolution, its first representative going to Lancaster, Pa., in 1773, afterward serving in the war and, later, living in Richmond, Va. Mr. Cohen's grandfather went to Baltimore in 1808. Two of the family served in the War of 1812. The father of Mendes Cohen, a banker in Baltimore, died in 1847, and the son, who had been under the instruction of a private tutor, directly entered the works of Ross Winans, the builder of early American locomotives. In 1851, he was made an Assistant Engineer on the Baltimore and Ohio Railroad, and served on the construction of the Broad Tree Tunnel; then, having been transferred to the Motive Power Department, he was engaged in the adaptation of wood-burning passenger locomotives to coal burning. He also worked out the method adopted for handling the traffic on the 10% temporary grade over the Kingwood Tunnel, a remarkable achievement in railroad operation.

In 1855, when only twenty-four years old, Mr. Cohen had already become known as an especially capable railroad official, and was made Assistant Superintendent of the Hudson River Railroad. He was with that Company until 1861, when he succeeded Gen. George B. McClellan as operating head of the Ohio and Mississippi Railroad, first as Superintendent and, later, as President and Superintendent. Soon after the close of the Civil War, he was engaged, for a short time, on special work for the Philadelphia and Reading Railway.

From 1868 to 1871 Mr. Cohen was Comptroller and Assistant to the President of the Lehigh Coal and Navigation Company. In 1872, he became President of the Pittsburgh and Connellsville Railroad, which was subsequently consolidated with the Baltimore and Ohio System. He retired in 1875 from official positions in connection with companies, but continued his practice in Baltimore as a Consulting Engineer.

Mr. Cohen is survived by his widow, Mrs. Jessie (Nathan) Cohen.

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* Memoir prepared by the following Committee: John Bogart and J. E. Greiner, Members, Am. Soc. C. E.

Mr. Cohen was a member of the Board appointed by the President, in 1894, to report on a route for the Chesapeake and Delaware Ship Canal. From 1893 to 1904, he was Chairman of the Baltimore Sewerage Commission. He was for twenty-one years the Secretary of the Maryland Historical Society, serving for nine years as its President. He was also Vice-President of the American Jewish Historical Society from 1897 until his death. His acquaintance with history, especially that of Maryland, was remarkable. He was a member of the Municipal Art Commission of Baltimore from 1892 until his death. For many years he was also an active member of the Board of Trustees of the Peabody Institute.

During many years Mr. Cohen was a strong and influential citizen in the city of his birth and long residence. He was consulted in all matters of great importance, and his judgment, always given after deliberate consideration, had great weight. He was a serious, religious man; socially of the best in Baltimore, in fact, a worthy representative of one of the old strong families of the United States.

Mr. Cohen was a man of strong character, with broad experience, not only in technical and engineering matters, but also in well-informed studies of affairs and conditions of business and social life. He was a man of much reserve, wise in conference, ready to consider and give weight to the suggestions of others, and did not hesitate to express his own judgment and opinion with clear statements of reasons for them. He was, therefore, a strong and valuable associate and adviser in the direction of business matters. This was particularly evident in the consideration of the problems arising in the affairs of a growing institution, such as the Society was during the years of his close connection with its management. He never lost interest in its welfare, nor forgot the responsibilities of a Past-President. He highly appreciated the honor of his election as its President.

Mr. Cohen became a Member of the American Society of Civil Engineers on December 4th, 1867. The Society, founded in 1852, had been inactive during the Civil War, but was kept alive by a small group of members, the officers elected in 1852 continuing their nominal functions. In 1867, the Society was resuscitated, James P. Kirkwood, one of the original members, becoming President in succession to James Laurie, who had held the office from 1852 to 1867. Mr. Cohen was elected to membership in that year, and, at the time of his death, had only one living associate of those who became members in 1867. The Society had only twenty-six members at the time of his election.

Although then only thirty-six years old, Mr. Cohen had already held responsible positions in railroad operation and management and

his well-informed judgment was at once of value in the building up and judicious expansion of the activities of the Society. He served as a Director in 1888, as Vice-President in 1890, and as President in 1892.

AUGUSTUS JAY DUBOIS, M. Am. Soc. C. E.*

Died October 19th, 1915.

Augustus Jay DuBois, the son of Henry Augustus DuBois and Catherine Helena (Jay) DuBois, who had six other children, was born at Newton Falls, Ohio, on April 25th, 1849. His father, who was of French Huguenot descent, received the degree of M.D. from Columbia College in 1830 and spent most of his life in the practice of medicine. His mother was a granddaughter of Chief Justice John Jay, who was also of French Huguenot descent.

Mr. DuBois prepared for college at the Hopkins Grammar School in New Haven, Conn., and then took the course in Civil Engineering at the Sheffield Scientific School of Yale University, from which he was graduated in 1869. Continuing there in advanced studies, he secured the degree of C. E. in 1870 and that of Ph.D. in 1873. He then spent 18 months at the Royal Mining Academy in Freiberg, Saxony, followed by a few months of surveying work in California and Connecticut.

During 1871-75 he made a special study of the then new science of Graphic Statics, the results of which were published in 1875, in two volumes, under the title "Elements of Graphical Statics and Their Application to Framed Structures." This was the first comprehensive work on the subject which appeared in the United States, and it was re-issued in revised editions in 1877, 1879, and 1883.

In 1875, Mr. DuBois was appointed Professor of Civil and Mechanical Engineering in Lehigh University, from which he was called, in 1877, to the chair of Mechanical Engineering at the Sheffield Scientific School, and, in 1884, he was appointed Professor of Civil Engineering there, a position which he filled until his death.

During his forty years of service as a teacher of Engineering, Professor DuBois was active in enriching the theory of the subject. He translated from the fourth edition of Weisbach's "Mechanics of Engineering", the sections "Hydraulics and Hydraulic Motors" (published in 1877), and "Theory of the Steam Engine" (1877); also Weyrauch's "Calculation of Iron and Steel Constructions" (1877), and Roentgen's "Principles of Thermodynamics" (1879). These books

* Memoir prepared by Mansfield Merriman, M. Am. Soc. C. E.

were issued at a time when literature on these subjects was scanty in the United States, and they were used extensively in engineering schools and by engineers, each of them passing through several editions. In 1883 appeared the first edition of his "Stresses in Framed Structures," a book in quarto form, which gave methods of computing stresses by both analytic and graphic processes and also contained fifteen folding plates of designs and working drawings of bridges. This book was widely used as a text in engineering schools, and it passed through twelve editions, each being revised and improved. The manuscript for an enlarged edition was completed shortly before his death.

In 1894 and 1895 there appeared three octavo volumes by Professor DuBois, entitled "Elementary Principles of Mechanics", one treating of kinematics, another of statics, and a third of kinetics. In 1901, these volumes were re-issued in revised quarto form as the first volume of "Mechanics of Engineering", the second volume being the twelfth edition of his "Stresses in Framed Structures."

Professor DuBois had contributed papers on roof trusses, retaining walls, and the steam engine to the technical journals; he had also taken part in discussions before the Society on flexure of beams, and had contributed papers on "The Weights of Bridges"* and on "The Strength of Columns".†

During 1889-94 he prepared and delivered six lectures entitled "Science and Faith", "Science and Immortality", "Science and Miracle", etc., which were published in the *Century Magazine* and other periodicals. These lectures were marked by originality of thought and beauty of style, and by the purpose to establish moral truths on the fundamental principles of mechanics; one of the last products of his pen was to summarize the conclusions of these lectures in an article in the *Yale Review* for July, 1913.

Professor DuBois was a hard worker, a clear and logical writer, and his books greatly advanced the interests of sound education in theoretical and applied mechanics. As a teacher, he was most successful, and especially was he insistent that students should acquire a thorough knowledge of fundamental principles. His successor, Professor John C. Tracy, in an obituary notice in the *Yale Alumni Weekly,* wrote as follows:

"A sympathetic interest, a ready wit, and a friendly unconventional manner won his students from the start. He was a clear and original thinker, and a keen but sympathetic critic. Breadth of culture and an unusual power of expression made him a brilliant and inspiring conversationalist. Underneath a quiet and undemonstrative exterior, there was a man chivalrous, sympathetic, always thoughtful of others, loyal,

* *Transactions,* Am. Soc. C. E., Vol. XVI (1887), p. 191; Vol. XVIII (1888), p. 170.

† *Transactions,* Am. Soc. C. E., Vol. XXVIII (1892), p. 69.

and wholly lovable. Only a few of his closest friends knew how, in his own quiet unostentatious way, he went about doing good, and to them he seemed an almost perfect type of Christian gentleman."

Professor DuBois rarely attended engineering meetings, seeming to feel somewhat awkward outside of the circle of his friends and students. In his college days he was a good chess player and a member of the Book and Snake Fraternity, but he took little interest in other social activities. He made six trips to Europe, for rest and relaxation during summer vacations, but he never had a Sabbatical year in whole or in part during his forty years of service as a teacher.

He was married, on June 23d, 1883, to Miss Adeline Blakesley, daughter of Arthur Blakesley, of New Haven, Conn. They had no children, and she survived him only seven months.

Professor DuBois was a member of the American Institute of Mining Engineers, the American Society of Mechanical Engineers, and the Society of Naval Architects and Marine Engineers, as well as several scientific academies and associations.

Augustus Jay DuBois was elected a Junior of the American Society of Civil Engineers on July 7th, 1875, and a Member on October 5th, 1892.

THEODORE NEWEL ELY, M. Am. Soc. C. E.*

Died October 28th, 1916.

Theodore Newel Ely was born in Watertown, N. Y., on June 23d, 1846. When he retired from the office of Chief of Motive Power of the Pennsylvania Railroad System on July 1st, 1911, he had lived through and participated in the most active period of railroad development that the world has ever seen, having begun his railroad career in 1868.

Mr. Ely was graduated from the Jefferson County (N. Y.) Institute in 1863, and from the Rensselaer Polytechnic Institute, of Troy, N. Y., with the degree of Civil Engineer, in the class of 1866.

After his graduation he had an opportunity to enter the United States Navy, the Navy Department having offered to accept the Institute's examinations as the equivalent of those at Annapolis and grant a full cadetship after a year of sea experience; another offer was that of a Professorship at Rensselaer. He preferred, however, active professional work.

After a brief experience in the office of the City Regulator (City Engineer) of Pittsburgh, Pa., he went to the Fort Pitt Foundry, where

* Memoir prepared by George L. Fowler, Esq., New York City.

he became associated with Gen. Thomas J. Rodman, the inventor of the well-known Rodman gun. The Government specifications for the metal for the guns were very exacting, and Mr. Ely here obtained his first experience in the investigation on a commercial scale of the qualities of metals. After the completion of this work, he was engaged for a short time in coal mining.

In 1868 he entered the Engineer Corps of the Pittsburgh, Fort Wayne and Chicago Railroad, at Pittsburgh. In July of the same year he accepted an offer of the position of Assistant Engineer on the Philadelphia and Erie Railroad, a part of the Pennsylvania Railroad System. In 1869, he was made Superintendent of the Middle Division, with headquarters at Renovo, Pa., and, in 1870, Assistant General Superintendent of the road. Some of the features of Mr. Ely's work during this period were re-alignments of the road, the rebuilding of practically every bridge on the Philadelphia and Erie, and the building of the docks and terminals at Erie Harbor, to take care of the road's prospective combined lake and rail traffic. During this period, also, the first steps were taken in the development of the Pennsylvania Railroad standards of materials and designs, and, in connection with Mr. Frank Thomson, who was Superintendent of the Eastern Division of the Philadelphia and Erie, he designed and made detailed drawings—the first issued at any point on the road—for a standard section of track. A sample mile of this track established its superiority over the previous construction, and, with the exception of the weight of the rail, its general features are still followed. He also conducted, in connection with Mr. William Robinson, an elaborate series of experiments with electric signals, which were fully developed later.

His duties as Assistant General Superintendent of the Philadelphia and Erie included the supervision of the Motive Power Department, of which he took direct charge in 1873, on his appointment as Superintendent of Motive Power.

In 1874, Mr. Ely was made Superintendent of Motive Power of the Pennsylvania Railroad Division, and, in 1882, General Superintendent of Motive Power of all the Lines East of Pittsburgh. His office in both these positions was at Altoona, Pa. Railroad conditions during Mr. Ely's administration at Altoona did not call for a radical change in the types of locomotives or cars, and their development, therefore, was largely in increasing the efficiency of the types then in general use by making important modifications and perfecting their details. He left his imprint on the locomotives of the country by the simple and clean-cut character of the designs developed at Altoona, which eliminated the useless and out-of-place ornamentation which was then a feature of locomotive construction. The same is true with regard to passenger equipment cars, for the improvement of which

Mr. Ely's good taste was largely responsible. Among the practices introduced during this period for increasing the efficiency and economy of operations, the following may be briefly mentioned.

At the time of Mr. Ely's appointment as Superintendent of Motive Power, the purchase of materials used was not controlled by specifications of any kind, the brand or reputation of the seller being depended on as a measure of quality. Recognizing that competitive purchasing required as its foundation a clear description of the article to be purchased, Mr. Ely obtained in 1875 permission to establish a Department of Tests, both physical and chemical, to develop the necessary specifications for the materials to be bought, and to examine the materials received to insure that they met the requirements of the specifications. Mr. Ely wisely selected young and ambitious men as the heads of the two branches of the Test Department, one being John W. Cloud and the other the late Charles B. Dudley, M. Am. Soc. C. E., and his firm support of his subordinates was largely responsible for the success of the new plan, as purchasing on the basis of specifications and tests alone naturally aroused strong opposition from many manufacturers, particularly those with established brands. In some cases, however, the manufacturers themselves were the strongest advocates of the general use of the specifications. The new Department speedily developed great value in the investigation of many of the technical questions which were constantly arising in railroad work, and it has made notable contributions in the line of scientific investigations covering a wide range of subjects.

Realizing the increasing responsibilities of the Mechanical Department, Mr. Ely encouraged the entrance into the service of young men of technical education, who were given a course of practical training in the shops under the same rules and regulations as the regular apprentices, their subsequent advancement depending on their development of the necessary qualifications.

The piecework system, which is now used on the whole Pennsylvania System, was introduced during the early part of Mr. Ely's administration. Another feature was the extensive use of committees for the study of important questions.

Mr. Ely's encouragement and the assistance given by his organization were large factors in the introduction and successful development of the automatic car coupler, the air brake, and other features of modern railroad practice.

An important undertaking was the establishment at Altoona, in 1889, of the Juniata Shops for building locomotives. These shops have been extended from time to time since to keep pace with the increasing size of modern locomotives, and still build the greater number of locomotives required by the Company.

Another notable practice established under Mr. Ely's direction was the pooling of the freight cars of all the lines of the Pennsylvania Railroad System, so far as repairs were concerned.

Incidents of this period were the active part which Mr. Ely took in the rebuilding of the road at Johnstown, after the flood in 1889; his work as a member of the Commission on Safe and Vault Construction, appointed by Act of Congress in 1890, for the purpose of improving the vault facilities of the Treasury Department; the exhibits of the Pennsylvania Railroad Company at the various International Expositions—the Centennial, Philadelphia, 1876; Paris, 1889; Columbian, Chicago, 1893; as well as the later ones, Louisiana Purchase, St. Louis, 1904; and Jamestown, 1907—which were prepared and carried out under his direction.

In 1893, Mr. Ely was appointed Chief of Motive Power of the Pennsylvania Railroad Lines East and West of Pittsburgh, with office at Philadelphia. Although in this position his duties were supervisory, he kept in close touch with the work of development of locomotives and passenger and freight cars to meet the changing conditions, both as to traffic and as to materials for car construction. He gave close study to and took an active part in the consideration of the questions of policy involved in the increases in the capacities of locomotives and cars, especially in freight service; the substitution of steel for wood in the construction of both freight and passenger cars; and the establishment of the general principles of the designs.

Other activities during this period were his work as Chairman of the Rail Committee of the System, charged with the improvement of the rail used; as a member of various committees of the American Railway Association; as a member of the Permanent Commission of the International Railway Congress; as President of the Eastern Railroad Association, dealing with patent matters; as a Trustee of the Philadelphia Commercial Museum, which is devoted to the development of the foreign trade of the United States; and as a member of the Pennsylvania State Commission on the Chestnut Tree Blight. He was also a Director, in the interest of the Pennsylvania Railroad, of the Pennsylvania and Cambria Steel Companies.

Mr. Ely was a member of the Institution of Civil Engineers of Great Britain, the American Society of Mechanical Engineers, the American Institute of Mining Engineers, and an Honorary Member of the American Institute of Architects.

He was also deeply interested in science and art, and took an active part in many organizations for their advancement, among them being the American Society for the Advancement of Science, the American Philosophical Society, the Franklin Institute, the Drexel Institute, the American Academy in Rome, Italy, of which he was

Vice-President, and the Philadelphia Orchestra Association, of which he was a Trustee.

Although ill health led to his retirement from active service with the Pennsylvania Railroad in 1911, it did not entirely deprive him of participation in the many affairs of life of which he was a part His keen analytical and judicial mind, trained and broadened by his studies and an experience covering almost the entire period of railroad development, made him a valued adviser in all the activities with which he came in contact.

Mr. Ely held the degree of Civil Engineer from Rensselaer Polytechnic Institute, as a graduate, and had conferred on him the honorary degrees of Master of Arts, Yale, 1897, and Doctor of Science, Hamilton College, 1904.

Among the social organizations with which he was identified were the Philadelphia and the Merion Cricket Clubs, of Philadelphia; the University, Century, and Engineers Clubs, of New York City; and the Metropolitan Club of Washington, D. C.; the New England Society of Pennsylvania, and the Sons of the American Revolution.

Mr. Ely is survived by his four children—Mrs. Charles L. Tiffany, of New York City, the Misses Gertrude and Henrietta Ely, of Bryn Mawr, Pa., and Carl B. Ely, Assoc. M. Am. Soc. C. E., Superintendent of the Bridge Department of the Bethlehem Steel Company, Steelton, Pa.

Mr. Ely was elected a Member of the American Society of Civil Engineers on March 2d, 1881, and served as a Director in 1892-93.

FRANK FIRMSTONE, M. Am. Soc. C. E.*

Died June 27th, 1917.

Frank Firmstone, the son of William and Mary Elizabeth Firmstone, was born on August 29th, 1846, at the Glendon Iron Works, near Easton, Pa., which his father had built in 1842. He received his early education at the Old Philips School in Easton. Later, he was sent to the Saunders Military Institute, in Philadelphia, Pa., where he was prepared for the Polytechnic College of Pennsylvania, and was graduated from that college as a Mining Engineer in June, 1865.

In November, 1865, Mr. Firmstone was employed as Levelman on surveys for the Wilmington and Brandywine Railroad, which position he held until March, 1866.

In January, 1867, he became associated with his father, as Assistant Superintendent, in the management of the Glendon Iron Works,

* Memoir prepared by the Secretary from information on file at the House of the Society.

remaining in this position until May, 1877, when his father died. Mr. Firmstone then took full charge of the Iron Works as General Manager, and retained this position until 1887, when he retired from active business. As Assistant Superintendent and as General Manager he made the designs and superintended the construction of all new work at the plant, including blast furnaces, etc., as well as repairs and alterations.

Subsequent to 1887, he became associated with the Cranberry Iron and Coal Company, serving for a number of years as its President and as a Director on the Boards of all its subsidiary companies until his death, which occurred at his home in Glendon, Pa., on June 27th, 1917.

He is survived by his brother, Mr. Harry Firmstone, of Longdale, Va.

Among other societies and clubs Mr. Firmstone was a member of the American Institute of Mining Engineers, and as such had contributed to its publications many interesting and valuable papers on blast furnace practice and on matters pertaining to the manufacture of pig iron, on which subject he was considered quite an authority.

He was also a member of the American Society for Testing Materials, American Forestry Association, Engineers' Club of New York City, Automobile Club of Philadelphia, Pa., Country Club of Northampton County, Pomfret Club of Easton, Pa., Northampton County Historical and Genealogical Society, and Trinity Protestant (Episcopal) Church, of Easton, Pa.

Mr. Firmstone was elected a Member of the American Society of Civil Engineers on August 7th, 1878.

JAMES WALTER GRIMSHAW, M. Am. Soc. C. E.*

DIED FEBRUARY 15TH, 1917.

James Walter Grimshaw was born at Manchester, England, on April 19th, 1852. In 1868, he was sent to Germany where he was a pupil of Carl Bach, a well-known civil engineer of Stuttgart, until 1870. He then entered the machine shops of E. Harn, of the same city, where he served an apprenticeship of one year. In 1871, Mr. Grimshaw entered the Stuttgart Royal Polytechnic School, from which he was graduated in Civil Engineering in 1874.

After his graduation, Mr. Grimshaw returned to England, where he was employed until 1877 in the Engineering Departments of Messrs.

* Memoir prepared by the Secretary from information on file at the House of the Society.

Sharp, Stewart and Company and Messrs. Ormerod, Grierson and Company, of Manchester. He was also engaged for a short time as Resident Engineer of the Netham Chemical Works at Bristol.

In 1877, Mr. Grimshaw went to Australia where he spent the greater part of his professional life. He first entered the service of the South Australian Government and received the appointment of Resident Engineer in the Harbors and Jetties Department, where he remained until 1880. He then entered the service of the Government of New South Wales, in the Rivers and Harbors Department, his first position being Resident Engineer of the Sydney Water Supply Works, in charge of the construction and maintenance of the canals, tunnels, aqueducts, etc.

About 1890, Mr. Grimshaw made the surveys for the proposed water supply for the City of Armidale, and supervised the completion and testing of the steel service reservoir for the Albury water supply.

On the completion of this work, he was appointed Resident Engineer on the Richmond River and Harbor Works, where two breakwaters and a training wall were being constructed. Subsequently, he was sent to Sydney as Resident Engineer over the works in Sydney Harbor and the South Coast District. In this position he had charge of extensive wharf construction and the conversion of Darling Island into a deep-water shipping depot.

In 1901, Mr. Grimshaw retired from the service of the Government of New South Wales and returned to England, where he resided until his death which occurred at Brighton, on February 15th, 1917.

He was a member of St. Stephen's Club, of London, and of the Institution of Civil Engineers of Great Britain.

Mr. Grimshaw was elected a Member of the American Society of Civil Engineers on November 7th, 1888.

WILLIAM HENRY HUNTER, M. Am. Soc. C. E.*

DIED FEBRUARY 27TH, 1917.

William Henry Hunter, the son of Henry Hunter, was born at Cork, Ireland, on June 16th, 1849. While he was quite young, his parents removed to England, and the boy was educated at private schools at his home in Sunderland and at the College of Physical Science, University of Durham, at Newcastle-on-Tyne.

In 1866, he entered the Engineering Workshops of the River Wear Commissioners where he remained until 1868, when he went into the office of the late Thomas Meik as a pupil.

* Memoir compiled by the Secretary from information on file at the House of the Society.

Having served his pupilage, Mr. Hunter, in January, 1870, was employed as Assistant in the office of Messrs. Meik and Nisbet, Harbor and Dock Engineers, of Edinburgh and Sunderland, and was engaged on harbor and dock work on the Rivers Wear, Blyth, and Coquet, at Burntisland and Ayr, and also on surveys for the Hylton, Southwick, and Monkwearmouth Railway, a branch of the North Eastern Railway.

From January, 1872, to September, 1873, he served as Resident Engineer on the construction of the Hylton, Southwick, and Monkwearmouth Railway in the County of Durham. He then accepted the position of Assistant Engineer on the reconstruction of the River Weaver Navigation, in the County of Chester, and remained in that position until 1882. The work consisted of enlarging and adapting the River Weaver for navigation by large barges and coasting vessels, thus giving commercial outlet to the interests engaged in the Cheshire salt trade.

From 1882 to 1885, Mr. Hunter was engaged as Chief Assistant to the late Sir Edward Leader Williams, on the surveys, preparation of designs, and Parliamentary contests, in connection with the project to connect the City of Manchester with the sea by a ship canal, and on work for the regulation and improvement of the River Dee in Cheshire and North Wales.

In 1885, when the Manchester Ship Canal Company was incorporated, Mr. Hunter was appointed Assistant Chief Engineer, and, as such, was directly and responsibly concerned with all the engineering operations connected with the actual design and construction of that waterway, including the Canal proper and all its auxiliary works.

In 1895, he was made Engineer and, in 1896, Chief Engineer of the Manchester Ship Canal, which position he held until 1910. During this time Mr. Hunter had charge of the maintenance of the entire project and carried out all additional works necessary to its development, including the extension of the great system of docks, railways, etc. It was during his incumbency as Chief Engineer, that the new Dock No. 9 was opened, with great ceremony, by His Majesty, the late King Edward VII, on July 5th, 1905.

In 1910, Mr. Hunter was appointed Consulting Engineer of the Manchester Ship Canal, which position he held until his death, which occurred at his home at Bank House, Woodley, Cheshire, on February 27th, 1917.

Mr. Hunter was recognized as the foremost English authority on canal and harbor construction, equipment, and operation. In 1898, he was appointed a member of the Comité Technique which was constituted by the French Government to consider and prepare plans, for the New Panama Canal Company, for the French Panama Canal, and, in 1905, at the request of the United States Government, he became a member of the Advisory Board of Consulting Engineers, appointed by

the President to consider and report on the plans for the American Panama Canal project, for which he strongly favored a sea-level, as against a lock, canal.

Mr. Hunter was a member of the Institution of Civil Engineers and frequently took part in the discussion of papers on canals and waterworks. He also presented a paper on the "Artificial Waterways in Great Britain",* before the International Engineering Congress, held in St. Louis, Mo., in 1904, under the auspices of the Society.

He was also a member of the Society of Arts and the Manchester Association of Engineers, and a Commissioner for the Navigation for the Upper Mersey. Mr. Hunter was of a deeply religious nature and was a devoted member of the Plymouth Brethren, frequently preaching and taking an active part in their meetings in Lancashire and Cheshire. A strenuous business man, he was noted for his gentleness and courtesy as husband, father, and host, and his love for children and their love for him was a frequent cause of remark. He is survived by his widow and three children.

Mr. Hunter was elected a Member of the American Society of Civil Engineers on February 7th, 1906.

WALTER KATTÉ, M. Am. Soc. C. E.†

Died March 4th, 1917.

Walter Katté was born in London, England, on November 14th, 1830. He was the son of Edwin Katté, and the grandson of Edwin Katté, a political refugee from Prussia during the reign of Frederick the Great. His mother, Isabel Chambers, was the granddaughter of John Chambers, a celebrated boat builder on the Thames, London. Walter Katté was educated in Kings College School, London, and, after his graduation, spent three years as an apprentice in the office of a civil engineer.

In 1849, he came to the United States and entered American railroad service as Clerk and Draftsman for the Chief Engineer of the Central Railroad of New Jersey, from Whitehouse to Easton, Pa. Later, he served as a Rodman and Assistant Engineer on the Belvidere and Delaware Railroad. In the early Fifties, he acted as Engineer for a land development company, and laid out the Town of Deerman, now Irvington-on-Hudson, N. Y.

During the three years following 1854, Mr. Katté was Chief Assistant Engineer on the Western Division of the Pennsylvania Railroad.

* *Transactions*, Am. Soc. C. E., Vol. LIV, Part F, p. 183.

† Memoir prepared by George W. Kittredge, M. Am. Soc. C. E.

Later, he acted, successively, as Resident Engineer of the Pennsylvania State Canals, as Assistant Engineer of the Pittsburgh, Fort Wayne and Chicago Railroad, and the Pittsburgh and Steubenville Railroad, until the breaking out of the Civil War.

In 1859, he was married, in Greensburg, Pa., to his first wife, Margaret Jack, who died in 1864, leaving one son.

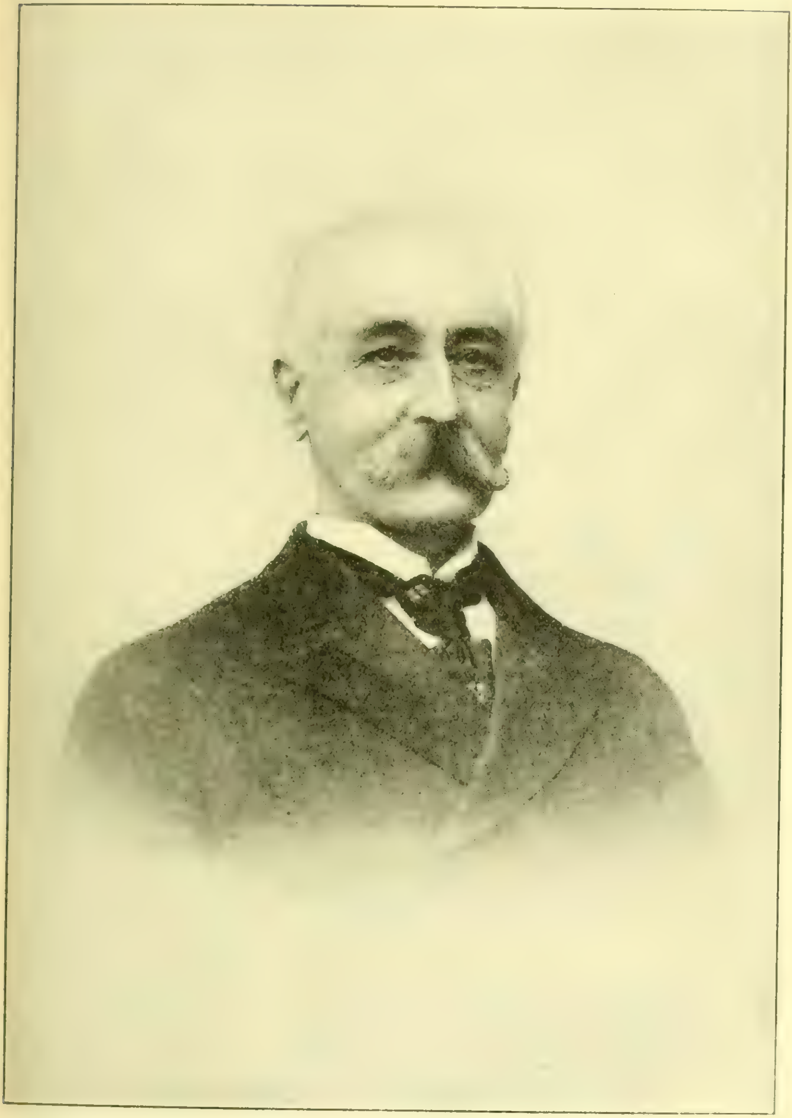
During 1861 and 1862, Mr. Katté served as a Colonel of Engineers in the Union Army, being commissioned from civil life to a staff position. He was assigned to bridge work in Washington, D. C., and at various points in Virginia and Maryland. He was the engineer in charge of the construction of the so-called "Long Bridge" over the Potomac River at Washington. While in this position, he had an experience with the great cavalry leader, General Philip Kearny. Returning one night from Washington to his regiment, which was quartered across the Potomac, the General sent his orderly ahead to demand passage over the bridge and received word that it was not in condition for traffic. He immediately rode his horse at full speed on to the bridge and, on being stopped peremptorily by Col. Katté, demanded an explanation as to why he was not permitted to proceed. Col. Katté quietly explained that a gap of 100 ft. or more in the bridge structure would prevent his further progress, except by swimming. Commenting editorially, a New York daily paper said:

"In the Civil War nothing was more important to the safety of Washington than keeping the railroads and bridges in its neighborhood in perfect shape. This work Col. Katté supervised efficiently. Though not won on the battlefield, his military title stood for active, energetic work in the interests of military defense."

In 1863, Col. Katté was engaged as Chief Engineer of the Lewiston Branch of the Pennsylvania Railroad, and, later, as Resident Engineer and Engineer of Bridges and Buildings on the Northern Central Railroad, from Baltimore, Md., to Elmira, N. Y.

Col. Katté compiled and wrote the first "Carnegie Pocket Companion" published, and, recently, at the request of the Carnegie Steel Company, wrote the history thereof, as follows:

"In 1865 to 1868, I was resident in Pittsburgh as Engineer and Secretary of the Keystone Bridge Company. In 1868, that company and the Union Iron Mills of Pittsburgh (Carnegie Bros., Kloman, Phipps & Co.) decided to enter the western field in competitive business and to establish an office and representative in Chicago for that purpose; I was chosen for that position. The Keystone Bridge Company had at that time already under contract the manufacture and erection of the superstructures of the Hannibal & St. Joe Railroad Company's bridge over the Missouri River, at Kansas City, and the Illinois Central Railroad Company's bridge over the Mississippi River, at Dubuque, Ia., also was negotiating contracts for bridges, later consummated, for Mississippi River at Keokuk, Ia., Louisiana, Mo., and St. Louis, Mo.



WALTER KATTÉ

"I proceeded to Chicago and opened there the western office of the Keystone Bridge Company and the Union Iron Mills, of Pittsburgh, Pa., and took personal charge, as agent and representative, of the field operations under these contracts.

"In 1870, negotiations for the great steel arch bridge over the Mississippi River at St. Louis were being actively promoted by Mr. Andrew Carnegie and myself, and finally consummated in the execution of a contract, signed by Capt. James B. Eads, as President of the Illinois & St. Louis Bridge Company, and by myself, on the part of the Keystone Bridge Company, under the terms of which, the Keystone Bridge Company undertook and obligated itself to perfect the mechanical details of the shop drawings of the superstructures, supply all materials for, and manufacture of, same, design plans for erection, and to erect it and assume all responsibility for the successful completion of the erection.

"I was assigned to take personal charge, as Resident Engineer, of said erection. As the responsibility for the successful consummation of same was of extreme gravity, I felt the paramount necessity of my personal presence on the work continuously, which, of course, resulted in the closing of my office in Chicago and the removal of same to St. Louis, which was effected early in 1871, and the joint Western office of the Keystone Bridge Company and the Union Iron Mills, of Pittsburgh, Pa., was opened under my charge at No. 211 Washington Avenue, St. Louis, Mo.

"About this time, or a little later, Mr. Thomas Carnegie suggested to me his desire to issue a handy 'Pocket Book' as a desirable assistant to Engineers and Architects in making proper selections, suited to their requirements, of the various products of the Union Iron Mills and asked me to compile the mss. for it, which I did. It was all written by my own hand from time to time in such leisure moments as were available, notwithstanding the pressing demands of my every day work, most of it done at home in evenings—and that's about all the early history that this little progenitor has to claim. It proved, however, a great success when issued, and there was, so Thomas Carnegie told me, a great demand for it, and he wrote me that he had received many letters from Engineers and Architects using it—highly extolling its usefulness and wondering why such a handy little *vade mecum* had not been issued long before."

While living in St. Louis, Col. Katté was married to Elizabeth Pendleton Britton, daughter of the Hon. James H. Britton, a prominent banker and later Mayor of that city.

After the completion of the St. Louis Bridge, Col. Katté was called to New York City to take the position of Chief Engineer of the New York Elevated Railroad Company, and from 1877 to 1880, he built the initial portions of the Third Avenue and Ninth Avenue Elevated Railroads, which were the first elevated steam railroads.

His next work was the construction of the New York, Ontario and Western Railroad, from Weehawken, N. J., to Middletown, N. Y.; then the building of the West Shore Railroad from New York City

to Buffalo, which was followed by the construction of the Jersey Junction Railroad, connecting the West Shore Railroad with the Pennsylvania Railroad, at Jersey City. This work occupied his time between 1880 and 1886.

In 1886, Col. Katté became Chief Engineer of the New York Central and Hudson River Railroad Company, which, at that time, absorbed the West Shore Railroad. His most important work while in the employ of this Company was the four-tracking and depressing of the tracks, in New York City, north of the Harlem River, this work being known as the Harlem Depression; the construction of the four-track steel viaduct in Park Avenue, New York City, and the four-track drawbridge over the Harlem River, which is still the largest drawbridge in existence. In 1898, Col. Katté resigned his position with the New York Central Company and, in his letter of resignation, stated: "The recent absorption of other railroad lines into the Vanderbilt System had so multiplied the duties of the office of the Chief Engineer, that he felt that a younger man was necessary for the work." In accepting his resignation, the Hon. Chauncey M. Depew, then President of the Railroad Company said: "Col. Katté is one of the foremost engineers in the world. He is still connected with the New York Central and Hudson River Railroad Company as Consulting Engineer, and will be as long as he lives."

Col. Katté was one of the original thirteen founders of the Western Society of Civil Engineers of which he only recently was elected an Honorary Member. He was also a member of the Institution of Civil Engineers, of Great Britain.

During his active engineering life, Col. Katté made frequent contributions to technical papers and to the *Transactions* of the National Engineering Societies. He published one of the first sets of standard specifications for railroad construction work, and had taken out several U. S. Patents, the one in most general use being his so-called "Three-tie rail joint."

Important daily papers at the time of Col. Katté's death were unanimous in their expression of the fine quality of his engineering work, one commenting editorially as follows:

"Col. Katté was a fine American, a great railroad builder, and had won first place among our civil engineers * * *. He knew and cared little about the devious ways of financing railroads; everything about construction and operation. Half a century of such activity fairly earned a period of repose. Col. Katté's later years were peaceful, calm, uneventful. He will live in the memory of his profession as a man who saw things clearly and who did things thoroughly. That is, from the practical viewpoint, the highest of encomiums."

Col. Katté enjoyed nearly nineteen years in quiet retirement. His health was excellent, with the exception of almost total deafness; his

mind alert and vigorous; his spirit strong and serene until the day of his death. He died at his home, in New York City, on March 4th, 1917, and is survived by a widow, two sons, and a daughter.

Col. Katté was elected a Member of the American Society of Civil Engineers on October 7th, 1868, and served as a Director in 1885 and 1889.

STANLEY ALFRED MILLER, M. Am. Soc. C. E.*

Died May 13th, 1917.

His many friends and associates were shocked beyond measure by the dreadful tragedy which culminated in the death of Stanley Alfred Miller and his assistant, Mr. Epaphroditus Hawkins, which took place about 42 miles from La Romana, Santo Domingo, on the morning of May 13th, 1917.

Stanley Alfred Miller was born in New Orleans, La., on May 27th, 1882, of one of the fine old families which have made his native State so famous by their ideals and accomplishments, and whose traditions he consistently maintained. He was educated at local schools and at the Louisiana State University, where he was one of the youngest students.

His professional work began like that of most engineers, as Rodman, Levelman, Instrumentman, etc. Then, he was Assistant Engineer on sewers, and on levee construction and railways. At the age of 19, however, Mr. Miller was Assistant Engineer to the Board of Public Works of Mobile, Ala., engaged in design and construction. As Assistant to various consulting engineers he was employed on the survey, design, and construction of sewers and water-works for Baton Rouge, La., Mobile, Ala., Dallas, Tex., and Ardmore and South McAlester, Okla. In 1904, he served as Resident Engineer on the Chihuahua and Pacific Railroad, and, in 1905, he was with S. Pearson and Son, Limited, Contractors, of London, England, on the construction of Port Works at Coatzacoalcos (now Puerto, Mexico), having been for a time in direct charge of the contractors' labor forces on the terminal yards and jetties. This will be recognized by all engineers as a position of great responsibility for a man of his age.

This was followed by his work for the Mexican Light and Power Company, near Necaxa, Mexico, in 1906, in local charge of the construction of Earth Dams Nos. 1 and 3, and Tunnel No. 1, where his infinite care and attention to every detail proved to be the source of much valuable information to his associates.

* Memoir prepared by Verne LeRoy Havens, M. Am. Soc. C. E.

While living in the semi-permanent camp near the dams, Mr. Miller was returning to his quarters after a midnight inspection of some of the work, when he stumbled over a severely wounded Mexican laborer. Having no assistance, and in spite of the fact that he was not a large man physically, he picked up the wounded man, carried him to his own cabin, and telephoned for a doctor. It was raining heavily, and the doctor was miles away attending to other patients, so Mr. Miller, by the light of a flickering and temporary arc lamp, dressed the abdominal wounds, the operation requiring many stitches. The man was doomed in any event, but Mr. Miller's efforts maintained the spark of life until his family could get to him.

Mr. Miller was engaged in the construction of a 7-mile sewer for Paducah, Ky., and paving and sewerage for Cairo, Ill., in 1906 and 1907. He was also Assistant Engineer on hydro-electric projects in California and Colorado in 1907 and 1908.

During the next three years, he was in charge of the Azua Irrigation Project, in Santo Domingo. On this work his duties were manifold and trying, but he completed them to his own satisfaction, and to those who knew him, the meaning of that statement will be clear, as he was his own most meticulous taskmaster. His work consisted of surveys and plans for, and reports on, the watering of 50 000 acres of sugar lands, and a hydro-electric development. After this, he took charge of the location of 55 miles of improved highways for Copiah County, Miss., and the construction of a portion of the work. He then joined the forces of the Uruguay Railway Company, and was placed in charge of railway location and construction, and the building of a reinforced concrete wharf, bridges, buildings, etc.

When visiting one of the camps under his supervision on this work, at Paloma del Sarandi, Mr. Miller found that one of the engineers had died in such a manner that local burial was impossible, and, in view of the refusal of the members of the party to assist him, he himself prepared the body for transportation. Then, with no food from the morning of one day until darkness of the second day, he traveled 60 miles, continuously, through the rainy night, in an ox-cart, secured help at the county seat, bought the ground, obtained authority for the burial, and organized a funeral, in order that the engineer might have the last rites due him, and that his parents, in far off Europe, might know where the body of their son lay.

On two separate occasions, Mr. Miller insisted on accompanying the writer on long trips without explaining his reasons for doing so, but the writer learned months afterwards that he had heard of threats against him for fancied errors in company policy.

From October, 1914, until December, 1916, Mr. Miller was engaged in general consultation practice at his home in Paducah, Ky. In this

capacity, he designed additional sewers for Paducah, reported on drainage projects in Kentucky, a hydro-electric plant in Mississippi, and rates for a public utility corporation. He also became well known throughout his adopted State by his earnest efforts, through the press and various organizations, to improve the status of the engineer, to eliminate engineering work from politics, and to further the general interest in good roads. He had built up a good practice, his work being continued during his last absence in Santo Domingo.

On December 9th, 1916, Mr. Miller sailed from New York for Central Romana, a sugar estate of the South Porto Rico Sugar Company, to "study a couple of rivers for hydro-electric and irrigation development", and lived in camp to push the work. This apparently was progressing nicely until the fateful Sunday morning of May 13th, 1917, when his camp was attacked shortly after daylight by a bandit with more than 100 men. The little party was quickly overpowered and led to their death, which took place about an hour afterward. One Spanish helper and a British negro were tied near Mr. Miller and Mr. Hawkins, but only the latter were executed. Mr. Miller explained to the bandit the nature of his work and its non-military character, but was told that "their fault was being Americans." His only plea, that he be allowed to write to his wife, was denied him. Mr. Miller's religious convictions were known only to those who knew him intimately, and it was his marvellous faith that the Divine Will would be done, and his maxim that "one must meet his portion without flinching", which undoubtedly formed the basis for his most extraordinary courage in facing death.

To those who stay behind, the loss is always keen and enduring, but the loss of Mr. Miller is felt with unusual regret. His rapid progress is proof to all of his professional value, and the younger engineers already counted him as a leader. His State will feel the loss of an able, analytical citizen, whose spirit was of the metal that tempers entire communities. The writer was associated with Mr. Miller on various works, and knows how much his friends will miss his counsel, and how much all those who knew him will miss the fine innate sense of justice, coupled with a knowledge of human weaknesses, which made his quietly expressed opinions grow on one like the conception of a fundamental law.

Mr. Miller was married to Miss Ann Bradshaw, of Paducah, Ky., on March 16th, 1911. Their first daughter, Miss Vauban Miller, died and was buried at sea, but their second daughter, Stanley Ann Miller, who was born just six weeks before her father went to Santo Domingo, together with her mother, survives him. He is also survived by his father and mother, Mr. and Mrs. W. R. Miller, his brother, Mr. W. R.

Miller, Jr., and his sister, Mrs. W. Molton Evans, all of Baton Rouge, La.

Mr. Miller was a member of New England Water Works Association, and the Boston Society of Civil Engineers. He was elected a Junior of the American Society of Civil Engineers on February 4th, 1902, an Associate Member on April 6th, 1909, and a Member on June 24th, 1916.

DAVID SIMSON, M. Am. Soc. C. E.*

Died December 16th, 1916.

David Simson was born at Bedrule House, Roxburghshire, Scotland, on November 9th, 1861. He was educated at St. Andrew's and Edinburgh University, and from September, 1877, to June, 1879, served as Apprentice Mechanical Engineer in the shops of Messrs. Douglas and Grant, at Kirkcaldy, Scotland, where he was engaged on the construction of large stationary engines, boilers, sugar and paper machinery, etc. From February, 1880, to January, 1882, he also worked as an Apprentice to Messrs. Carfrae and Belfrage, Land Surveyors, of Edinburgh, Scotland, and was engaged in surveying, the construction of roads, sewers, etc.

Having served his apprenticeship, Mr. Simson was employed, from January, 1882, to July, 1884, by Messrs. J. Waddell and Son, Railway Contractors, of Edinburgh, as Contractor's Engineer in charge of the construction of the eastern half of the Edinburgh Suburban Railway. This work involved the deviation of streets, sewers, etc., and the crossing of the new railway over and under existing railways and canals.

From August to November, 1884, Mr. Simson traveled over the greater part of the United States and Canada, in order to study American methods of railroad construction. On his return to his home, he was engaged, from January to June, 1885, by the creditors of Messrs. L. and K. Macdonald, Contractors, to finish the construction of the Killin Railway, in Perthshire, Scotland. One of the most important parts of this work was the construction of a viaduct over the River Dochart. This viaduct consisted of five skew spans of about 50 ft. each, the arches being of concrete. This is the first instance in which concrete was used for arches of this magnitude in railway construction in Great Britain.

In June, 1885, Mr. Simson entered the employ of John Strain, Civil Engineer, and was engaged in making surveys and preparing

* Memoir prepared by the Secretary from information on file at the House of the Society.

drawings for the various railway construction projects on which the firm was working. He also assisted Mr. Strain in giving evidence before Parliamentary Commissions in London, and in 1887-88, served as Resident Engineer on the construction of the Kilbirnie Branch of the Lanarkshire and Ayrshire Railway, for which he had previously made the surveys and drawings.

In September, 1888, Mr. Simson resigned his position with Mr. Strain, and went to South America where he was appointed Chief Assistant Resident Engineer on the Mountain Division of the Buenos Aires and Valparaiso Transandine Railway from Mendoza. This Division covered 100 miles of extremely difficult mountain railway construction, especially in connection with the gorge of the Mendoza River in which the roadbed had to be blasted from the sides of the precipice. Mr. Simson remained in this position until November, 1891, at which time the line was constructed and open as far as Rio Blanco, and work on the Summit Tunnel had been commenced.

He then returned to Scotland, but, in October, 1892, he was appointed Chief Engineer of the Western Railway of Havana, in Cuba, which position he held until June, 1896. During his incumbency, all the old timber trestles along the line of this road were replaced by steel structures, and the railroad was extended 10 miles to Pinar del Rio, into the heart of the best tobacco district on the island.

In November, 1896, Mr. Simson entered the service of the Buenos Aires Western Railway Company as Chief Engineer, with headquarters at Buenos Aires. In September, 1897, he was appointed Acting Manager, and in September, 1898, General Manager of the Company, which position he held until December, 1906. On his retirement, the employes of the Railway Company presented him with a silver model of a locomotive in token of their affection and respect.

From January to March, 1907, Mr. Simson was engaged in Chile and Bolivia, in making a report on the affairs of the Antofagasta and Bolivia Railway. In April of that year, he retired from the active practice of engineering and returned to England to make his home.

Subsequently, he was elected a member of the Board of Directors of the Buenos Aires Western, the Antofagasta and Bolivia, and the Great Western of Brazil Railways, but he was obliged to retire from the latter two, in 1912 and 1913, respectively, on account of pressure of other work. He had also been elected, in November, 1907, a Director of the Buenos Aires Great Southern Railway Company, and was made Chairman of the Board in September, 1910. He was also a Director of the Buenos Aires Southern Dock Company and of the London and River Plate Bank.

At the time of his death, which occurred suddenly by heart failure at his home at Ickleford Manor, Hitchin, Herts, Mr. Simson was actively engaged in war relief work in his district.

He was a member of the Institute of Civil Engineers of Great Britain, and a "Caballero" of the Second Class of the Spanish Order of Military Merit, having been decorated for services rendered the Spanish Government in conducting armored trains and repairing the Western Railway of Havana under fire during the Cuban Rebellion of 1896.

He was widely versed in South American affairs, and his death will be deeply felt by the Anglo-Argentine Colony of which he was one of the best known and most valued members. He is survived by his widow and one son.

Mr. Simson was elected a Member of the American Society of Civil Engineers on January 8th, 1902.

EDWARD CLINTON TERRY, M. Am. Soc. C. E.*

Died April 6th, 1908.

Eli Terry, of Plymouth, Conn., began to make mantel clocks in 1814, and these soon drove other styles out of the market. A few of these mantel or shelf clocks, with wooden wheels and bearing the legend "Warranted if well used", are still sometimes found in Connecticut. His son Eli continued the manufacture of clocks at Terryville, Conn., a village which he founded about 1825, and his grandson James inherited the business, later turning it into the manufacture of locks for trunks. James Terry was of a highly mechanical turn of mind; he constructed a pipe organ and a water motor in his house, and also made a working model of a steam road vehicle which walked on four legs like a horse.

Edward Clinton Terry, the youngest child of James and Elizabeth (Hollister) Terry, was born at Terryville on December 10th, 1850. As a lad, he had the advantage of learning much from his father regarding machinery and hydraulic motors. He prepared for college at the High School, in Hartford, Conn., and took the Civil Engineering course at the Sheffield Scientific School of Yale University, from which he was graduated in 1871. As a student, his work was well and faithfully done, and among his classmates he was regarded as a clear thinker, fond of philosophical discussions, and as an excellent chess player.

During 1871-73, Mr. Terry worked as a Rodman on the Easthampton Branch of the Connecticut River Railroad, and was also engaged

* Memoir prepared by Mansfield Merriman, M. Am. Soc. C. E.

on surveys of the reservoirs for the water supply of Hartford, Conn. He then spent much time in the study of water meters, and took out patents for both a rotary and a piston meter. The first of these had a wabbling disk which caused the rotation of an axis that acted on the recording apparatus; this he called the "Undine Meter", the name being taken from a German story which was read by his class in the Freshman year. In 1880, he became connected with the Hartford Meter Company, with which he was associated until 1889, at first as General Manager and Secretary, and, later, as its proprietor.

In 1890, Mr. Terry was one of the organizers of the Farmington River Power Company, and, as its Engineer, he designed and constructed a plant for generating electric power on the Farmington River and transmitting it a distance of 11 miles to Hartford. This work included a dam 500 ft. long and 18 ft. high, the hydro-electric power plant, and the transmission line. It delivered at first 800 h.p. to the Hartford Electric Light Company, and, later, this was increased to 1 300 h.p. This plant is said to have been the first long-distance transmission of power by electricity in America. He was Secretary of the Company and Manager of its engineering work for several years, and it was during this period that modern methods were first used in generating electric energy from water power and in transmitting it to Hartford.

About 1888, Mr. Terry became interested in steam turbines and, in the following ten years, took out patents for three of high speed and three of low speed. In 1906, he organized a company for the manufacture of low-speed turbines, of which he was President until his death. Two of these turbines were built first in New York City at the Waterside Stations of the Edison Company, and were used to drive boiler-feed pumps; in 1917 there are 24 such steam turbine-driven pumps in use at these Stations. Altogether, about 3 300 Terry turbines have been made and put into operation since 1906.

Mr. Terry had a wide reputation as a man of mechanical genius and high business capacity. Although rather retiring in manner and of a modest disposition, he was cheerful in temperament. Those who became his friends always remained such, admiring his independence of thought, his power of keen analysis of political and philosophical problems, and his sterling character.

In college, Mr. Terry was a member of the Book and Snake Fraternity and of the Sheffield Chess Club. In Connecticut, he was a member of the Hartford Club and of the Golf Club. In politics, he was always an independent voter. In 1901, he went to Europe, but his inclination for travel was never great, and he preferred the quiet of his home and the conversation of his family and friends. He died

on April 6th, 1908, at his residence in Hartford, Conn., following a week's illness of pneumonia.

Luther W. Burt, Esq., formerly City Engineer of Hartford, writes as follows:

"Terry always met an acquaintance with a cordial greeting and a pleasant smile. His conversation was always informing and never intrusive. I cannot remember that he ever spoke a disparaging word of any one. He was upright, studious, and self-centered. He had, I think, few intimates, but he generously recognized the services of friends and employees."

Henry W. Sargent, Esq., Vice-President of Sargent and Company, of New Haven, Conn., writes:

"Graduating as a Civil Engineer, Terry's inherited mechanical instincts led his imaginative and reflective mind into mechanical and electrical fields, which he enjoyed and enriched. With this taste and with capacity for practical invention, he possessed executive qualities, in a nice command of others, with patience to lead his associates to reason and work with him in the direction of his choice. Such power, strengthened by personal morality and a high ethical sense, makes for a leader of men.

"Terry's nervous energy too early wore out a not too strong physique, and he was called to his fathers at a time when his exertions had just placed his manufacturing company on the firm foundation of developing and supplying what was wanted in a field of prime movers all its own."

James Shepard, Esq., of New Britain, Conn., who was Attorney for Mr. Terry in taking out his patents for water meters and steam turbines, writes as follows:

"My acquaintance with 'Clinton Terry' (as he was generally called), began about 1878, when he became my client. He was one of the most interesting clients I ever had, and it was always a pleasure to meet him. He generally had some useful information to impart, oftentimes upon matters other than the main subject of our interviews. He was cheerful, energetic, diligent, persistent, and generally successful in all his undertakings. He was free to express his opinion as to what he wanted, but he always did so in a courteous and pleasant manner. He was a man of good habits, strictly honest, and prompt in all business matters. His inventions were always fully worked out, even as to details, and reduced to working drawings, before making applications for patents. He was liked by all with whom he came into contact, and his influence upon others was of an elevating character.

"His persistency enabled him to overcome obstacles that would have discouraged other men. As an illustration of this, he came into possession of the first clock model made by his great-grandfather, Eli Terry, after a lengthy controversy and the payment of $1 000 for the clock."

Mr. Terry was married on February 28th, 1872, to Miss Louise Ellen Webster, of Terryville, Conn., who survives him. They had two children: Charles Webster Terry, who died in 1886; and James Terry, who was graduated from the Sheffield Scientific School in 1895, succeeded his father as President of the Terry Steam Turbine Company, and died in 1917.

Edward Clinton Terry was elected a Member of the American Society of Civil Engineers on February 6th, 1895.

EDWARD THOMAS WRIGHT, M. Am. Soc. C. E.*

Died March 29th, 1917.

Edward Thomas Wright was born at Elgin, Ill., on June 30th, 1851. His parents were Paul R. Wright and Emily Grace (Harvey) Wright. He attended the public school at Geneva, and, later, at Cobden, Ill., where the family moved when he was about nine years of age. After graduating from the public schools, he had a private tutor for a year and then attended the Elgin Academy.

After leaving the Academy, Mr. Wright took up the study of engineering, and, in 1872, entered the office of Messrs. Cleveland and French, prominent engineers of Chicago, Ill., and was employed by them on work at Indianapolis, St. Paul, and Chicago, including the South Chicago Drainage, for the development of an industrial district.

Mr. Wright moved to Los Angeles, Cal., in the latter part of 1874 and established himself as a Civil Engineer and Surveyor, maintaining his office up to the time of his death, on March 29th, 1917. He had served three terms as County Surveyor, and was a hydraulic engineer of ability, having done much effective work in developing underground waters and constructing irrigation systems.

In 1873 Mr Wright was married to Lucy Nicholson, who died in 1899. He leaves two sons, George A. and Charles N., who follow his profession, and a widow, Mrs. Capitola B. (Wenzel) Wright, whom he married in March, 1912.

He was a man highly respected for his sterling worth and honesty; ever ready to help others, and loved by many friends.

Mr. Wright was elected a Member of the American Society of Civil Engineers on February 3d, 1886.

* Memoir prepared by W. D. Larrabee, M. Am. Soc. C. E.

WILLIAM HERBERT HYDE, Assoc. M. Am. Soc. C. E.*

Died April 15th, 1917.

William Herbert Hyde, the son of Peter Lowe and Anne Elizabeth (Copcutt) Hyde, and the grandson of the late John Copcutt, of Yonkers, N. Y., was born at the latter place on December 15th, 1874. He was of English descent, one great-grandfather having been an officer (Captain) in the Revolution and another in the War of 1812. Both his grandfather and his father were engaged in the mahogany and hardwood export and import business in New York City.

Mr. Hyde received his education at the public and grammar schools, and the Yonkers High School, and, in 1895, entered the employ of the New York Central and Hudson River Railroad as Chainman in the Chief Engineer's Department, advancing to Assistant Engineer of the Maintenance of Way Department in 1898. From that time until 1901 he served on location and construction with the Erie Transit Company and the Panama Railroad; on topographical work and estimates for the Barge Canal, New York State; and as Assistant Engineer in the Atlantic City, N. J., Water Department.

During 1901 Mr. Hyde was engaged in surveying and contracting work, being associated with Pittsburgh interests. In April, 1902, he entered the contracting field, forming a partnership with his father-in-law, Mr. W. F. Patterson, of Pittsburgh, Pa., and specialized in shaft and underground improvement work.

During the last ten years, he had conducted his business alone, his operations including coal shafts in Pennsylvania, West Virginia, Nova Scotia, and Alberta, Canada. He built many fine concrete-lined pumping stations underground and successfully completed contracts for such firms as the New River Company, The Berwind-White Company, The Ragland Coal Company, and others of equal standing. He often contributed articles of merit to leading engineering and mining journals, and has been freely quoted as an authority in works on coal mine engineering.

Mr. Hyde died at Scarboro, W. Va., on Sunday, April 15th, 1917, from a bullet wound received while attempting to place an unruly workman under arrest.

He was married on February 15th, 1903, to Miss C. Virginia, daughter of Walter F. Patterson, of Pittsburgh, Pa., who, with four young children, survives him.

Mr. Hyde was a member of the Sons of the Revolution. He was elected a Junior of the American Society of Civil Engineers on April 30th, 1901, and an Associate Member on June 4th, 1902.

* Memoir prepared by S. M. Van Loan, M. Am. Soc. C. E.

SAMUEL FORSYTHE THOMSON, Assoc. M. Am. Soc. C. E.*

Died January 30th, 1917.

Samuel Forsythe Thomson was born in Charleston, S. C., on June 5th, 1872. His parents were Samuel and Elizabeth Craig Forsythe Thomson, natives of County Antrim, near Belfast, Ireland, who came to Charleston at an early age, were married and made their home there. His father was a prominent merchant in Charleston for many years, and was well and favorably known throughout that section of the South, being prominently identified with many of the activities of his community.

Mr. Thomson first attended a public school in Charleston, and, after finishing the course there, entered the Charleston High School, from which he was graduated with honor in June, 1889. A life-long friend of the family testifies to the fact that he was a favorite alike with teachers and pupils. He then attended a school at Boylston and Tremont Streets, Boston, Mass., where he was prepared for a higher education. Mr. Thomson entered the Massachusetts Institute of Technology in September, 1890, and specialized in civil engineering.

During the summer vacations of 1891 and 1892 he was engaged on engineering work with the firms of Aspinwall and Lincoln and E. A. W. Hammatt, Civil Engineers, of Boston, Mass., and during the summer of 1893 attended the summer school of the Institute of Technology at Keeseville, N. Y. In the summer of 1894 he was employed with John W. Arnold, United States Marshal, of Chicago, Ill., and later for a short period was with the late Col. George E. Waring, Jr., in the Department of Street Cleaning, New York City.

During 1895-96 Mr. Thomson took an additional course at the Institute of Technology in railroad engineering. He was graduated from the Institute in June, 1896, with the degree of Bachelor of Science in Civil Engineering.

After graduation he re-entered the service of the Department of Street Cleaning of New York City under the Commissioner, Col. George E. Waring, Jr., and until May, 1898, was a member of his personal staff. During his administration as Commissioner, Col. Waring gained an international reputation by reorganizing the Department, introducing many reforms, and placing it on an efficient and business-like basis. Mr. Thomson acted as Chief Assistant to the Master Mechanic, and had charge under his supervision of the design and erection of two steel storage dumps, also the general design and repairs to city property, such as stables, dumps, and scows, and of snow removal above 59th Street, Manhattan. He also supervised the introduction of the system of separation of city refuse throughout

* Memoir prepared by Robert Ridgway and George A. Taber, Members, Am. Soc. C. E.

the entire city, and of the construction and operation of the first crematory used for the economical utilization of city wastes for the development of power.

From May, 1898, to April, 1903, Mr. Thomson was the Assistant to the Professor of Civil and Military Engineering at the United States Military Academy, West Point, N. Y., and then entered the service of the Commission on Additional Water Supply for the City of New York, this commission having been appointed to investigate all available sources for an additional water supply for the city. Mr. Thomson was in charge of the New York office of the Long Island Department, under Walter E. Spear, M. Am. Soc. C. E., which Department was engaged in the investigation of the available underground supply on Long Island.

On July 23d, 1903, he was appointed an Assistant Engineer in the Engineering Department of the Rapid Transit Railroad Commission, of New York City, and was assigned to the Fifth Division under Robert Ridgway, M. Am. Soc. C. E., Division Engineer, on the construction of the rapid transit tunnels under the East River from South Ferry, Manhattan, to Joralemon Street, Brooklyn, which tunnels are now in operation. He had much to do with the triangulation and other survey work in connection with the alignment of the tunnels, and for a time was in charge of the general office management of the Division. He resigned on March 5th, 1906, to accept an appointment as Assistant Engineer in the service of the Board of Water Supply of the City of New York, of which J. Waldo Smith, M. Am. Soc. C. E., was Chief Engineer, and was assigned to the Wallkill Division of the Northern Aqueduct Department by the Department Engineer, Robert Ridgway. The Wallkill Division, with headquarters at New Paltz, included about 12 miles of Catskill Aqueduct construction, which aqueduct was designed for a daily capacity of 500 000 000 gal. The most notable feature of construction was the deep rock pressure tunnel, 4.4 miles long, under the Wallkill Valley, this tunnel being located at a maximum depth of about 520 ft. below the hydraulic gradient. Other features of construction were the south half of the Bonticou grade tunnel through the Shawangunk Mountains, about 6¾ miles of cut-and-cover aqueduct on the slope of these mountains and on the east side of the Wallkill Valley, and the blow-off chamber and conduit leading from the aqueduct to the Wallkill River. The cost of construction of the work on the Division was approximately $5 500 000. For a time Mr. Thomson was in charge of the organization of the division office, and was instrumental in establishing methods and a routine which proved to be most efficient in carrying out the work. Following the transfer to other work of James F. Sanborn, Assoc. M. Am. Soc. C. E., Division Engineer, Mr. Thomson, on February 1st, 1912, was made Acting Division Engineer in his place, and so remained until the practical completion of the work.

Of his record on the Wallkill Division, Ralph N. Wheeler, Assoc. M. Am. Soc. C. E., now in charge of the Northern Aqueduct Department, writes:

"His work as an organizer of a working force and of systems of routine, records, filing, etc., was unusual. In no other division of this department were records kept so faithfully and well, supplies, equipment, etc., so completely accounted for, and large and small matters relating to the work so completely recorded. I shall always believe that this was due very largely to Mr. Thomson's faithful oversight. His loyalty to his superiors, consideration for his subordinates and in fact his every-day devotion to his work, even in the small and often disagreeable details, are the characteristics for which we shall always remember him."

On June 15th, 1914, he left the service of the Board of Water Supply and accepted employment with the Kingsbridge Contracting Company, of New York City, as Civil Engineer. He was identified with the operations of that company, particularly in the construction of the large outlet sewer which was built in connection with the Seventh Avenue Rapid Transit Subway, from that avenue to the Hudson River, the cost of this sewer being about $500 000. He was also connected with the construction of the large trunk sewer in East 41st Street, as well as with other projects carried out by that company.

On his return home from the excursion during the Annual Meeting of the American Society of Civil Engineers, on January 18th, 1917, Mr. Thomson was stricken with the illness which resulted in his death twelve days later.

A friend, of many years standing, who knew the personal as well as the business side of his character, writes:

"To those who knew him intimately many of his finer qualities were alone revealed. His friendship possessed those priceless and rare characteristics which endeared him to all who were privileged to be counted among his friends. Its endurance was unquestionable, and at all times his active and unqualified support to a friend could always be depended upon. His sense of honesty was also of the highest quality. In all his dealings the moral obligation in a business transaction was as important and binding to him as any of the legal provisions, and this quality characterized all of his acts through life. He was in all respects an ideal friend, in the finer sense of the word, and none could desire or have a better."

Another associate of his in his later work says:

"I consider him one of the finest characters I have ever met. In the three years of our acquaintance and friendship we were together considerably during business hours and out of them. At all times and under all conditions, many of which were often trying, his pleasant smile and cheery way never left him. He was very careful not to hurt any one's feelings, and never spoke disparagingly or unkindly of any

person; he generally had some explanation or excuse for the failings of the one who might be under discussion and who had not measured up to the full stature of a man."

Loyalty to his principles and to his friends, honesty, humanity, and steadfastness of purpose were prominent among the underlying qualities of his nature, and the character built on such a foundation was one which his friends will long remember with affection and pride.

Mr. Thomson was married on September 29th, 1898, to Miss Jennie A. Milton of Danvers, Mass., who with one daughter, Elizabeth, survives him. He was a member of the New England Water Works Association, the Municipal Engineers of the City of New York, and the Brooklyn Engineers Club. He was much interested in the work of the Fourth Unitarian Church, of Brooklyn, N. Y., of which he was a member.

Mr. Thomson was elected an Associate Member of the American Society of Civil Engineers on January 3d, 1906.

LEWIS ROBERTS POMEROY, Assoc. Am. Soc. C. E.*

Died May 7th, 1917.

Lewis Roberts Pomeroy was born at Port Byron, N. Y., on February 5th, 1857, but his early life was spent in Milwaukee, Wis., where his father was in business. In his fifteenth year he came East, and attended school at the Irving Institute in Tarrytown, N. Y., for one year. His subsequent education was obtained entirely by his own efforts, for, from this time, he supported himself by filling various clerical positions until, in 1880, he became Secretary and Treasurer of the Suburban Rapid Transit Company, which position he filled until 1886.

From 1886 to 1895 Mr. Pomeroy was with the Carnegie Steel Company, during which time he introduced basic boiler steel for locomotives and special forgings. He was a painstaking student of locomotives and locomotive details, and exerted a strong and effective influence in improving such materials and in the detailed design of locomotives. Subsequently, he was engaged in the same kind of work jointly for the Cambria Steel Company and the Latrobe Steel Company. From 1899 to 1902, he was Assistant General Manager of the Schenectady Locomotive Works, and from 1902 to 1908, he was representative in the railway field for the General Electric Company.

Engineers owe Mr. Pomeroy a debt of gratitude for his studies in the electrification of steam railroads, for his knowledge and experience

* Memoir prepared by George M. Basford, Esq., New York City.

enabled him to reveal many important phases of this problem. His earlier work in connection with electric power in railroad and manufacturing shops was of great importance in methods of modernization, and he was undoubtedly among the highest authorities on railroad shop layouts and equipment.

Mr. Pomeroy's next appointment was as Assistant to the President of the Safety Car Heating and Lighting Company, and, subsequently, he was appointed Chief Engineer of the Railway and Industrial Divisions of J. G. White and Company. In June, 1914, he was appointed Manager of the New York Sales Office of the United States Light and Heating Company, but later opened an office as Consulting Engineer in New York City.

Mr. Pomeroy had been in the railway and railway supply business for more than thirty-five years, and many officials and engineers, especially those connected with railroads and railroad developments, will sadly miss his friendly helpfulness. His own wide engineering knowledge came through painstaking study, experience, and contact. His kindly nature was attracted to young men who were struggling to succeed; his natural desire was to encourage and aid them, and to lead them to improve and to take important places in the world's work. Would that we had more men inspired as he was with the desire to help others! It became an important part of Mr. Pomeroy's life-work to discover young engineers of ability and to bring them to the front, and this will be to him an enduring monument.

His death, which was very sudden, occurred at his home in East Orange, N. J., on May 7th, 1917. He is survived by his wife and two children.

Mr. Pomeroy was elected an Associate of the American Society of Civil Engineers on April 2d, 1890.

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"THE DISTRIBUTION OF STRESSES IN MITERING LOCK-GATES, WITH SPECIAL REFERENCE TO THE GATES ON THE PANAMA CANAL." HENRY GOLDMARK.

"AIR TANKS ON PIPE LINES." MINTON M. WARREN.

"THE CAPE COD CANAL." WILLIAM BARCLAY PARSONS. (To be presented Oct. 3d, 1917.)

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OF THE

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OF

CIVIL ENGINEERS

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SEPTEMBER, 1917

Edited by the Secretary, under the direction of the Committee on Publications.

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The House of the Society is open from 9 A. M. to 10 P. M. every day, except Sundays, Fourth of July, Thanksgiving Day, and Christmas Day.

HOUSE OF THE SOCIETY—220 WEST FIFTY-SEVENTH STREET, NEW YORK.

TELEPHONE NUMBER........................1446 Circle.
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Vol. XLIII. SEPTEMBER, 1917. No. 7.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PROCEEDINGS

SOCIETY AFFAIRS

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MINUTES OF MEETINGS

OF THE SOCIETY

September 5th, 1917.—The meeting was called to order at 8.30 P. M.; President Pegram in the chair; Chas. Warren Hunt, Secretary; and present, also, 78 members and 5 guests.

The minutes of the meetings of May 16th and June 6th, 1917, were approved as printed in *Proceedings* for August, 1917.

A paper by Floyd A. Nagler, Jun. Am. Soc. C. E., entitled "Obstruction of Bridge Piers to the Flow of Water", was presented by the author and illustrated with lantern slides. The Secretary read communications on the subject from Messrs. Mansfield Merriman, E. W. Lane, A. J. Wiley, and R. D. Goodrich, and the paper was discussed orally by Messrs. F. H. Frankland and C. E. Fowler.

The Secretary read a letter from the representatives of the widow of the late Stanley A. Miller, M. Am. Soc. C. E., who was murdered by bandits in Santo Domingo, on May 13th, 1917. This letter re-

quested the Society to pass resolutions urging upon the State Department prompt and vigorous prosecution of a claim by Mrs. Miller and her infant daughter against the Republic of Santo Domingo for a money indemnity. There was enclosed with this letter a copy of the petition which has been filed with the State Department of the United States requesting interposition with the Republic of Santo Domingo in order that Mrs. Miller may be compensated for the death of her husband, and reciting a complete history of the case.

On motion, duly seconded, the President was authorized to appoint a suitable committee to draft a communication to the State Department of the United States in the form of resolutions, setting forth the sense of the meeting and memorializing the State Department and recommending the action sought by Mrs. Miller.

The Secretary announced the following deaths:

William Harry Arnold, of New York City, elected Member, May 1st, 1907; died August 13th, 1917.

Charles Lee Crandall, of Ithaca, N. Y., elected Junior, June 7th, 1876; Member, October 5th, 1892; died August 25th, 1917.

Frank Firmstone, of Easton, Pa., elected Member, August 7th, 1878; died June 27th, 1917.

James Edgar Jenkins, of New York City, elected Associate Member, December 5th, 1906; Member, March 14th, 1916; died July 5th, 1917.

David Wendel Spence, of College Station, Tex., elected Member, October 1st, 1913; died June 29th, 1917.

Joel Manning Howard, of Ogdensburg, N. Y., elected Associate Member, June 4th, 1913; died May 22d, 1917.

Charles Marvin Everest, of Rochester, N. Y., elected Fellow, November 1st, 1892; died July 22d, 1917.

Adjourned.

ANNOUNCEMENTS

The House of the Society is open from 9 A. M. to 10 P. M., every day, except Sundays, Fourth of July, Thanksgiving Day, and Christmas Day.

FUTURE MEETINGS

October 3d, 1917.—8.30 P. M.—This will be a regular business meeting. A paper by William Barclay Parsons, M. Am. Soc. C. E., entitled "The Cape Cod Canal", will be presented for discussion.

This paper was printed in *Proceedings* for August, 1917.

October 17th, 1917.—8.30 P. M.—At this meeting two papers will be presented for discussion, as follows: "Detention Reservoirs with Spillway Outlets as an Agency in Flood Control", by H. M. Chittenden, M. Am. Soc. C. E.; and "Hydraulic Phenomena and the Effect of Spreading Flood Water in the San Bernardino Basin, Southern California", by A. L. Sonderegger, M. Am. Soc. C. E.

These papers are printed in this number of *Proceedings*.

November 7th, 1917.—8.30 P. M.—A regular business meeting will be held, and a paper entitled, "The Subsidence of Muck and Peat Soils in Southern Louisiana and Florida", by Charles W. Okey, Assoc. M. Am. Soc. C. E., will be presented for discussion.

This paper is printed in this number of *Proceedings*.

SEARCHES IN THE LIBRARY

In January, 1902, the Secretary was authorized to make searches in the Library, upon request, and to charge therefor the actual cost to the Society for the extra work required. Since that time many searches have been made, and bibliographies and other information on special subjects furnished.

The resulting satisfaction, to the members who have made use of the resources of the Society in this manner, has been expressed frequently, and leaves little doubt that if it were generally known to the membership that such work would be undertaken, many would avail themselves of it.

The cost is trifling compared with the value of the time of an engineer who looks up such matters himself, and the work can be performed quite as well, and much more quickly, by persons familiar with the Library.

In asking that such work be undertaken, members should specify clearly the subject to be covered, and whether references to general books only are desired, or whether a complete bibliography, involving search through periodical literature, is desired.

It sometimes happens that references are found which are not readily accessible to the person for whom the search is made. In that case the material may be reproduced by photography, and this can be done for members at the cost of the work to the Society, which is small. This method is particularly useful when there are drawings or figures in the text, which would be very expensive to reproduce by hand.

A list of 989 bibliographies made in the Library, giving the cost of each, was published in Vol. LXXX of *Transactions*.

Since October 1st, 1916, the Library of the American Society of Civil Engineers has ceased to exist, as such, having been merged with the Libraries of the Mining, Mechanical, and Electrical Engineers, and become a part of the Library of the United Engineering Society. There were 67 000 accessions, which were not duplicates, turned over to that Library.

Hereafter, therefore, requests for searches should be addressed to the Librarian, United Engineering Society, 29 West 39th Street, New York City.

PAPERS AND DISCUSSIONS

Members and others who take part in the oral discussions of the papers presented are urged to revise their remarks promptly. Written communications from those who cannot attend the meetings should be sent in at the earliest possible date after the issue of a paper in *Proceedings*.

All papers accepted by the Publication Committee are classified by the Committee with respect to their availability for discussion at meetings.

Papers which, from their general nature, appear to be of a character suitable for oral discussion, will be published as heretofore in *Proceedings*, and set down for presentation to a future meeting of the Society, and on these oral discussions, as well as written communications, will be solicited.

All papers which do not come under this heading, that is to say, those which, from their mathematical or technical nature, in the opinion of the Committee, are not adapted to oral discussion, will not be scheduled for presentation to any meeting. Such papers will be published in *Proceedings* in the same manner as those which are to be presented at meetings, but written discussions only will be requested for subsequent publication in *Proceedings* and with the paper in the volumes of *Transactions*.

The Board of Direction has adopted rules for the preparation and presentation of papers, which will be found on page 429 of the August, 1913, *Proceedings*.

LOCAL ASSOCIATIONS OF MEMBERS OF THE AMERICAN SOCIETY OF CIVIL ENGINEERS

San Francisco Association, Organized 1905.

J. D. Galloway, President; E. T. Thurston, Secretary-Treasurer, 57 Post Street, San Francisco, Cal.

. The San Francisco Association of Members of the American Society of Civil Engineers holds regular bi-monthly meetings, with banquet, and weekly informal luncheons. The former are held at 6 P. M., at the Palace Hotel, on the third Tuesday of February, April, June, August, October, and December, the last being the Annual Meeting of the Association.

Informal luncheons are held at 12.30 P. M., every Wednesday, and the place of meeting may be ascertained by communicating with the Secretary.

The by-laws of the Association provide for the extension of hospitality to any member of the Society who may be temporarily in San Francisco, and any such member will be gladly welcomed as a guest.

(Abstract of Minutes of Meeting)

August 21st, 1917.—The meeting was called to order at the Palace Hotel; Vice-President Brunnier in the chair; E. T. Thurston, Secretary; and present, also, 66 members and guests.

Messrs. Bush, Judell, and Wolf were appointed as the Entertainment Committee for the October meeting.

For the Committee on the Frickstad Case, Professor Charles D. Marx reported that Mr. Frickstad had been entirely vindicated and reinstated and that a full report on the matter would probably be presented by the Committee at the next meeting.

For the Committee on Civil Service Laws, Mr. J. Newman gave a résumé of a lengthy report embodying a comparative analysis of the civil service laws of the United States, the State of California, and the Cities of San Francisco, Los Angeles, and Oakland. This report was referred to the Board of Directors with authority to publish it.

For the Committee on the Discussion of Papers published in *Proceedings,* Mr. H. D. Dewell read a report recommending against sectional meetings for the discussion of Society papers on account of the small attendance at such meetings and the tendency to detract from the attendance and interest of the regular meetings. The report recommended that occasional regular meetings be devoted to the discussion of Society papers to be selected by the Board of Directors. Discussion of the recommendations of this report will be in order at the October meeting.

For the Committee on Building Construction Safety Orders, the Secretary reported that the final meeting of the General Committee had been held, and that the Industrial Accident Commission would shortly issue a code of safety rules and regulations governing building construction throughout the State.

The Secretary reported the receipt of a communication from Director J. V. Davies requesting the assistance of the Association in securing all available information regarding highways applicable for

military purposes in District No. 13, for the use of the Council of National Defense.

A paper by Walter C. Howe, M. Am. Soc. C. E., entitled "State Highway Construction in California under Commission Supervision", was presented by the author, who illustrated his remarks with stereopticon views.

Adjourned.

Colorado Association, Organized 1908.

Robert Follansbee, President; L. R. Hinman, Secretary-Treasurer, 1400 West Colfax Avenue, Denver, Colo.

The meetings of the Colorado Association of Members of the American Society of Civil Engineers (Denver, Colo.) are held on the second Saturday of each month, except July and August. The hour and place of meeting are not fixed, but this information will be furnished on application to the Secretary. The meetings are usually preceded by an informal dinner. Members of the American Society of Civil Engineers will be welcomed at these meetings.

Weekly luncheons are held on Wednesdays at 12.30 P. M., at Daniel's and Fisher's.

Visiting members are urged to attend the meetings and luncheons.

Atlanta Association, Organized 1912.

Paul H. Norcross, President; Thomas P. Branch, Secretary-Treasurer, Georgia School of Technology, Atlanta, Ga.

The Association holds its meetings at the University Club, Atlanta, Ga. Regular monthly luncheon meetings are held to which visiting members of the Society are always welcome.

Baltimore Association, Organized 1914.

Mason D. Pratt, President; Charles J. Tilden, Secretary-Treasurer, The Johns Hopkins University, Baltimore, Md.

(Abstract of Minutes of Meeting)

May 2d, 1917.—The meeting was called to order at 9 P. M., at the Merchants Club; President Bush in the chair; C. J. Tilden, Secretary; and present, also, 24 members and 1 guest.

The minutes of the meeting of March 21st, 1917, were read and approved.

The Secretary-Treasurer was instructed to send out to letter-ballot the proposed revision of the Constitution of the Association, which was submitted to the meeting of March 21st, 1917, and had received the formal approval of the Board of Direction of the Society on April 17th, 1917.

The report of the Committee appointed to study and report on the relations of Local Associations to the Society was presented.

On motion, duly seconded, it was ordered that the report be received and the course of action outlined therein be followed.

The Secretary presented a letter from Thomas C. Desmond, Assoc. M. Am. Soc. C. E., calling attention to the opportunity for engineers in the engineering regiment to be attached to the proposed Roosevelt

Division. On motion, duly seconded, the matter was laid on the table.

The Secretary-Treasurer submitted his report of receipts and expenditure for the year ending May 1st, 1917. He also reported that the Association now had 58 members, and that the total membership of the Society in Maryland was 121.

In answer to a question, the Secretary stated that invitations to join the Association had been sent to the remaining members of the Society in Maryland, and that a small number had accepted. On motion, duly seconded, the report was accepted.

The Dinner Committee, Messrs. Warren and Whitman (Mr. Greiner absent), made a brief report.

The following officers were elected: President, Mason D. Pratt; Vice-President, Ezra B. Whitman; Secretary-Treasurer, Charles J. Tilden; Directors, W. H. Dorsey, A. H. Hartman, W. D. Janney, Jenks B. Jenkins, W. W. Pagon, and W. F. Strouse.

A vote of thanks was given to the Dinner Committee, and also to the Secretary-Treasurer.

President Pratt took the chair and addressed the meeting briefly.

Messrs. Pitts, Warren, and Whitman were appointed a committee to report on the advisability of holding monthly meetings of the Association, with or without a luncheon or dinner.

On motion, duly seconded, it was decided that meetings be held at least once every two months.

Adjourned.

Cleveland Association, Organized 1914.

W. J. Watson, President; George H. Tinker, Secretary-Treasurer, 516 Columbia Building, Cleveland, Ohio.

(Abstract of Minutes of Meeting)

June 30th, 1917.—The meeting was called to order; President Watson in the chair; George H. Tinker, Secretary; and present, also, 18 members.

Correspondence was read from the St. Louis Association, and also from Chas. Warren Hunt, Secretary of the Society, concerning the rejection by the Board of Direction of the Association's request for the appointment of a Committee on Rivers and Harbors.

President Watson announced that 23 000 duplicate volumes from the Library of the Society had been turned over to the Association for placing in the Library of the Cleveland Engineering Society, and that the latter had accepted the custody of the books.

President Watson announced that Messrs. Robert Hoffman, D. Moomaw, and K. H. Osborn, had been appointed as delegates to the Inter-Society Committee.

In the matter of the report of the Committee of the Board of Direction on Relations of Local Associations, the Secretary was directed to request written discussion from members of the Association and to forward such discussions to the Secretary of the Society.

The Secretary read a letter from Director J. V. Davies requesting the Association to assist in securing information concerning highways

in District No. 6 for the Council of National Defense. After discussion, it was voted that the Executive Committee appoint a committee of three to take charge of the collection of the information, and that this committee be empowered to call on other members of the Association and of the Society in the Sixth District for assistance.

Adjourned.

Detroit Association, Organized 1916.

T. A. Leisen, President; Clarence W. Hubbell, Secretary, 2334 Dime Bank Building, Detroit, Mich.

The regular meetings of the Association are held on the second Friday of December, April, and October, the last being the Annual Meeting.

District of Columbia Association, Organized 1916.

A. P. Davis, President; John C. Hoyt, Secretary-Treasurer, U. S. Geological Survey, Washington, D. C.

Duluth Association, Organized 1917.

F. E. House, President; Walter G. Zimmermann, Secretary, Wolvin Building, Duluth, Minn.

The regular meetings of the Association are held monthly. The time and place of meeting are not fixed, but this information will be furnished on application to the Secretary. The Annual Meeting is held on the third Monday of May.

Illinois Association, Organized 1916.

C. F. Loweth, President, Chicago, Ill.

The regular meetings of the Association are held on the second Monday of March, June, September, and December, the last being the Annual Meeting. The hour and place of meeting are not fixed, but this information will be furnished on application to the President.

Louisiana Association, Organized 1914.

W. B. Gregory, President; Charles W. Okey, Secretary, Tulane University, New Orleans, La.

The regular meetings of the Association are held at The Cabildo, New Orleans, La., on the first Monday of January, April, July, and October.

Nebraska Association, Organized 1917.

Frank T. Darrow, President; Homer V. Knouse, Secretary-Treasurer, 115 City Hall, Omaha, Nebr.

Regular meetings of the Association are held on the first Saturday of each month, except July and August, and at such places as may be appointed from time to time by the Executive Committee. The Annual Meeting is held in Lincoln, Nebr., on the second Friday in January.

It is probable that frequent luncheons will be held in Omaha, in addition to the monthly meetings, at which visiting members will be

welcomed. The place of meeting may be ascertained by communicating with the Secretary.

Northwestern Association, Organized 1914.

George L. Wilson, President; Ralph D. Thomas, Secretary, 508 South First Street, Minneapolis, Minn.

Philadelphia Association, Organized 1913.

Samuel T. Wagner, President; C. W. Thorn, Secretary, 1313 South Broad Street, Philadelphia, Pa.

The regular meetings of the Association are held at the Engineers' Club of Philadelphia, 1317 Spruce Street, on the first Monday in January, April, and October, the last being the Annual Meeting.

Portland, Ore., Association, Organized 1913.

J. P. Newell, President; J. A. Currey, Secretary, 194 North 13th Street, Portland, Ore.

St. Louis Association, Organized 1914.

J. A. Ockerson, President; Gurdon G. Black, Secretary-Treasurer, 34 East Grand Avenue, St. Louis, Mo.

The meetings of the Association are held at the Engineers' Club Auditorium. The Annual Meeting is held on the fourth Monday in November. The time of other meetings is not fixed, but this information will be furnished on application to the Secretary.

San Diego Association, Organized 1915.

W. J. Gough, President; J. R. Comly, Secretary-Treasurer, 4105 Falcon Street, San Diego, Cal.

Seattle Association, Organized 1913.

Joseph Jacobs, President; Carl H. Reeves, Secretary-Treasurer, 444 Henry Building, Seattle, Wash.

The regular meetings of the Association are held at 12.15 P. M., on the last Monday of each month, at The Frye Hotel.

Southern California Association, Organized 1914.

H. Hawgood, President; Wilkie Woodard, Secretary, 435 Consolidated Realty Building, Los Angeles, Cal.

The Southern California Association of Members of the American Society of Civil Engineers (Los Angeles, Cal.) holds regular bi-monthly meetings, with banquet, at Hotel Clark, on the second Wednesday of February, April, June, August, October, and December, the last being the Annual Meeting of the Association.

Informal luncheons are held at 12.15 P. M. every Wednesday, and the place of meeting may be ascertained from the Secretary.

The by-laws of the Association provide for the extension of hospitality to any member of the Society who may be temporarily in Los Angeles, and any such member will be gladly welcomed as a guest at any of the meetings or luncheons.

(Abstract of Minutes of Meeting)

August 15th, 1917.—The meeting was called to order at the Hotel Clark; President Hawgood in the chair; M. K. Barnard acting as Secretary.

The minutes of the meeting of April 11th, 1917, were read and approved, and reports of Committees were called for. The following new members were introduced: Messrs. E. G. Sheibley, Charles H. Wondries, and E. L. Adams.

Mr. H. W. Dennis, of the Programme Committee, announced that Mr. T. D. Allin, Commissioner of Public Works, of Pasadena, had extended an invitation to the Association to inspect the new Municipal Sewage Disposal Plant, on September 8th, 1917.

Mr. George G. Anderson, Chairman of the Committee on Relations of Local Associations, submitted a written report of that Committee. On motion, duly seconded, it was decided to postpone action on the report until the next meeting of the Association.

Mr. Anderson gave an interesting talk on the recent flood in the Colorado River, illustrating his remarks with lantern slides. During the course of this address, attention was called to the remarkably correct predictions of the Weather Bureau Official at Denver, which were of invaluable service to those in charge of the flood protection work in the Imperial Valley.

On motion, duly seconded, the Secretary was instructed to write to Mr. F. H. Brandenburg, in charge of the Weather Bureau Office, in Denver, complimenting him on the accuracy of his predictions and expressing the appreciation of the Association of the value of his work.

Adjourned.

Spokane Association, Organized 1914.

J. C. Ralston, President; B. J. Garnett, Secretary, City Hall, Spokane, Wash.

The regular meetings of the Association are held on the second Friday of each month, except July and August. The hour and place of meeting are not fixed, but this information will be furnished on application to the Secretary.

Visiting members are invited to attend the meetings and luncheons.

Texas Association, Organized 1913.

John B. Hawley, President; J. F. Witt, Secretary, Dallas, Tex.

Utah Association, Organized 1916.

George L. Swendsen, President; H. S. Kleinschmidt, Secretary-Treasurer, 306 Dooly Building, Salt Lake City, Utah.

The Annual Meeting of the Association is held on the first Wednesday in April. The time of other meetings is not fixed, but this information will be furnished on application to the Secretary.

PRIVILEGES OF ENGINEERING SOCIETIES EXTENDED TO MEMBERS OF THE AMERICAN SOCIETY OF CIVIL ENGINEERS

Members of the American Society of Civil Engineers will be welcomed by the following Engineering Societies, both to the use of their Reading Rooms, and at all meetings:

American Institute of Electrical Engineers, 25 West Thirty-ninth Street, New York City.

American Institute of Mining Engineers, 25 West Thirty-ninth Street, New York City.

American Society of Mechanical Engineers, 25 West Thirty-ninth Street, New York City.

Assoçiacão dos Engenheiros Civis Portuguezes, Lisbon, Portugal.

Australasian Institute of Mining Engineers, Melbourne, Victoria, Australia.

Boston Society of Civil Engineers, 715 Tremont Temple, Boston, Mass.

Brooklyn Engineers' Club, 117 Remsen Street, Brooklyn, N. Y.

Canadian Society of Civil Engineers, 176 Mansfield Street, Montreal, Que., Canada.

Civil Engineers' Society of St. Paul, St. Paul, Minn.

Cleveland Engineering Society, Chamber of Commerce Building, Cleveland, Ohio.

Cleveland Institute of Engineers, Middlesbrough, England.

Dansk Ingeniorforening, Amaliegade 38, Copenhagen, Denmark.

Detroit Engineering Society, 46 Grand River Avenue, West, Detroit, Mich.

Engineering Association of Nashville, Commercial Club Building, Nashville, Tenn.

Engineers and Architects Club of Louisville, 1412 Starks Building, Louisville, Ky.

Engineers' Club of Baltimore, 6 West Eager Street, Baltimore, Md.

Engineers' Club of Kansas City, E. B. Murray, Secretary, 920 Walnut Street, Kansas City, Mo.

Engineers' Club of Minneapolis, 17 South Sixth Street, Minneapolis, Minn.

Engineers' Club of Philadelphia, 1317 Spruce Street, Philadelphia, Pa.

Engineers' Club of St. Louis, 3817 Olive Street, St. Louis, Mo.

Engineers' Club of Toronto, 96 King Street, West, Toronto, Ont., Canada.

Engineers' Club of Trenton, Trent Theatre Building, 12 North Warren Street, Trenton, N. J.

Engineers' Society of Northeastern Pennsylvania, 415 Washington Avenue, Scranton, Pa.

Engineers' Society of Pennsylvania, 31 South Front Street, Harrisburg, Pa.

Engineers' Society of Western Pennsylvania, 568 Union Arcade Building, Pittsburgh, Pa.

Florida Engineering Society, J. R. Benton, Secretary, Gainesville, Fla.

Institute of Marine Engineers, The Minories, Tower Hill, London, E., England.

Institution of Civil Engineers, Great George Street, Westminster, S. W., London, England.

Institution of Engineers of the River Plate, Calle 25 de Mayo 195, Buenos Aires, Argentine Republic.

Institution of Naval Architects, 5 Adelphi Terrace, London, W. C., England.

Junior Institution of Engineers, 39 Victoria Street, Westminster, S. W., London, England.

Koninklijk Instituut van Ingenieurs, The Hague, The Netherlands.

Louisiana Engineering Society, State Museum Building, Chartres and St. Ann Streets, New Orleans, La.

Memphis Engineers' Club, Memphis, Tenn.

Midland Institute of Mining, Civil and Mechanical Engineers, Sheffield, England.

Montana Society of Engineers, Butte, Mont.

North of England Institute of Mining and Mechanical Engineers, Newcastle-upon-Tyne, England.

Oregon Society of Civil Engineers, Portland, Ore.

Pacific Northwest Society of Engineers, 803 Central Building, Seattle, Wash.

Rochester Engineering Society, Rochester, N. Y.

Sociedad Colombiana de Ingenieros, Bogota, Colombia.

Sociedad de Ingenieros del Peru, Lima, Peru.

Societe des Ingenieurs Civils de France, 19 rue Blanche, Paris, France.

Society of Engineers, 17 Victoria Street, Westminster, S. W., London, England.

Svenska Teknologforeningen, Brunkebergstorg 18, Stockholm, Sweden.

Tekniske Forening, Vestre Boulevard 18–1, Copenhagen, Denmark.

Vermont Society of Engineers, George A. Reed, Secretary, Montpelier, Vt.

Western Society of Engineers, 1737 Monadnock Block, Chicago, Ill.

ACCESSIONS TO THE UNITED ENGINEERING SOCIETY LIBRARY

(From July 11th to August 9th, 1917)

DONATIONS*

The statements made in these notices are taken directly from the books themselves, and this Society is not responsible for them.

WAR-SHIPS:

A Text-Book on the Construction, Protection, Stability, Turning, Etc., of War Vessels. By Edward L. Attwood. 6th ed. N. Y. and Lond., Longmans, Green and Co., 1917. 338 pp., 209 illus., 9 x 6 in., cloth. $4.00.

This book has been written in response to suggestions made by senior naval officers taking the course in Naval Architecture at the Royal Naval College, Greenwich. An attempt has been made to treat the subject from the naval officers' standpoint, and certain parts have been treated with a view of meeting their special requirements.

SHIPYARD PRACTICE AS APPLIED TO WARSHIP CONSTRUCTION.

By Neil J. McDermaid. 2d ed. N. Y. and Lond., Longmans, Green & Co., 1917. 332 pp., 153 illus., 9 x 6 in., cloth. $4.00.

A course of lectures given to Cadets of Naval Instruction at the Royal Naval College, Devonport. Describes the actual operations to be performed during the construction and outfitting of a warship.

NAVAL ARCHITECTURE.

By Cecil H. Peabody. 4th ed., rev. N. Y., John Wiley & Sons, Inc.; Lond., Chapman & Hall, Ltd., 1917. 641 pp., 218 illus., 1 diagram, 9 x 6 in., cloth. $7.50.

Intended to give a consistent and connected statement of the commonly accepted theory of naval architecture. Intended for naval architects and shipbuilders, as well as for students. Discussions of certain devices for checking rolling of ships, such as Frahm's anti-rolling tanks and the steadying gyroscopes of Schlick and Sperry, have been introduced in this edition.

THE MODERN GAS TRACTOR:

Its Construction, Utility, Operation, and Repair: A Practical Treatise Covering Every Branch of Up-to-Date Gas Tractor Engineering, Driving, and Maintenance in a Non-Technical Manner, Considers Fully all Types of Power Plants and Their Components, Methods of Drive and Speed Changing Mechanisms, Describes Design and Construction of all Parts, Their Installation and Adjustment, as well as Practical Application of Tractors in the Field. By Victor W. Page. 2d ed., rev. and enl. N. Y., The Norman W. Henley Publishing Co., 1917. 32 + 504 pp., 225 illus., 7 x 5 in., cloth. $2.00.

This treatise is intended to bridge the gap between the purely technical work and the manufacturers' instruction book dealing with one specific construction.

* Unless otherwise specified, books in this list have been donated by the publishers.

GAS AND FUEL ANALYSIS FOR ENGINEERS:

A Compend for Those Interested in the Economical Application of Fuel. Prepared Especially for the Use of Students at the Massachusetts Institute of Technology. By Augustus H. Gill. 8th ed., rev. N. Y., John Wiley & Sons, Inc.; Lond., Chapman & Hall, Ltd., 1917. 145 pp., 20 illus., 7 x 5 in., cloth. $1.25.

An attempt to present in a concise yet clear form the methods of gas and fuel analysis involved in testing the efficiency of a boiler plant. The directions for using the bomb calorimeter have been made to agree more closely with present usage in this edition, and minor corrections and additions have been made throughout the text.

GAS-LIGHTING AND GAS-FITTING:

A Pocket Book for Gas Companies, Gas Engineers, and Gas Fitters, for Manufacturers of Gas Fixtures and Dealers in Gas Appliances, for Gas Consumers, Architects, and Builders, Health Officers and Sanitary Inspectors. By Wm. Paul Gerhard. 4th ed. (Van Nostrand Science Series.) N. Y., D. Van Nostrand Co., 1913. 190 pp., 3 illus., 6 x 4 in., boards. 50 cents. (Gift of the author and publisher.)

COMPRESSED AIR:

Theory and Computations. By Elmo G. Harris. 2d ed., rev. and enl. N. Y., McGraw-Hill Book Co., Inc.; Lond., Hill Publishing Co., Ltd., 1917. 15 + 192 pp., 32 illus., 9 x 6 in., cloth. $2.00.

Designed to present the mathematical treatment of the problems in the production and application of compressed air. In the present edition, errors and ambiguities have been eliminated and new matter has been added.

UNITED STATES ARTILLERY AMMUNITION:

3 to 6-in. Shrapnel Shells, 3 to 6-in. High Explosive Shells and Their Cartridge Cases. By Ethan Viall. N. Y., McGraw-Hill Book Co., Inc.; Lond., Hill Publishing Co., Ltd., 1917. 98 pp., 171 illus., 12 x 9 in., cloth. $2.00.

This work is intended to give shop men, engineers, and manufacturers an accurate knowledge of the sizes, tools, shop work, and gauges for the more commonly used United States shells and cartridge cases. The descriptions are in minute detail and accompanied by numerous dimensioned drawings.

MACRAE'S BLUE BOOK:

America's Greatest Buying Guide; Vol. 8, 1917. Chic. and New York, MacRae's Blue Book Co. 1 344 pp., 11 x 9 in., cloth. $10.00.

Contents: Catalogue Section; Address Section; Representatives' Index; Classified Material Section; Comprehensive Trade Name Index; Miscellaneous Data Section; Standard List Prices of Building Materials and Iron and Steel Products; A Net Discount Computer.

EXPORT TRADE DIRECTORY, 1917-1918.

Compiled by B. Olney Hough. N. Y., American Exporter, Johnston Export Publishing Co. 537 pp., 1 map, 9 x 6 in., cloth. $5.00.

Contents: Export Merchants in the United States; Manufacturers' Export Agents, Managers of Export Departments and Export Brokers; Leading Bankers Engaged in Foreign Exchange Business; Foreign Exchange Brokers; Marine Insurance Companies in New York City; Foreign Freight Forwarders; Some Export Trucking Companies in New York City; Steamship Services to Foreign Ports; How to Ship to Foreign Markets; Consuls of Foreign Countries in the United States; United States Consular and Commercial Representatives in Foreign Countries; Associations for the Promotion of Export Trade.

OFFICE ORGANIZATION AND MANAGEMENT.

By Carl C. Parsons. Chic., LaSalle Extension University, 1917. 14 + 313 pp., 59 illus., 1 diagram, 8 x 6 in., leather. $2.50.

Treats of organization, management, layout, equipment, methods, systems, records, forms, employes, etc. Based on observation of the methods used in the offices of various large companies.

INTRODUCTION TO THE RARER ELEMENTS.

By Philip E. Browning. 4th ed., rev. N. Y., John Wiley & Sons, Inc.; Lond., Chapman & Hall, Ltd., 1917. 10 + 250 pp., 6 illus., 1 pl., 9 x 6 in., cloth. $2.00.

The first edition of the book was prepared by the author from material used in a short lecture course at Yale University, as a convenient handbook in the introductory study of the rarer elements. Numerous changes and additions have been made in the present edition, in order to bring it up to date.

PHOTOGRAPHY IN COLOURS.

By George Lindsay Johnson. New and rev. ed. N. Y., E. P. Dutton & Co., 1917. 302 pp., 22 illus., 14 pl., 8 x 5 in., cloth. $2.00.

Intended to supply the information necessary for successful work, with any of the recognized methods of photography in colors. In this edition errors have been corrected, descriptions of the Raydex color process, Gaumont's method of cinematography in colors, and Carrara's method of reproducing autochromes on paper, have been added, and chapters are included on art in color photography and on photomicrography in color.

CHEMICAL DISCOVERY AND INVENTION IN THE TWENTIETH CENTURY.

By Sir William A. Tilden. N. Y., E. P. Dutton & Co.; Lond., George Routledge and Sons, Ltd. (preface, 1916). 16 + 487 pp., 150 illus., 11 por., 9 x 6 in., cloth. $3.50.

A semi-popular account of modern chemical discovery, covering both theory and applications. Contents: Chemical Laboratories and the Work Done in Them; Modern Discoveries and Theories; Modern Applications of Chemistry; Modern Progress in Organic Chemistry.

CITY PLANNING PROGRESS,

In the United States, 1917. Compiled by the Committee on Town Planning of the American Institute of Architects. Edited by George B. Ford, Assisted by Ralph F. Warner. Washington, D. C., The Journal of the American Institute of Architects. 8 + 207 pp., 113 illus., 11 x 8 in., paper. $1.50.

A report on what has been accomplished or is projected in city planning in all cities in the United States of more than 25 000 inhabitants, and in a few cities and towns with a smaller population where the work is of special interest. Little attention has been devoted to housing, as a separate book on this subject is being prepared. Particular stress has been laid on the economic and engineering aspects of city planning. The report is based on questions sent out by the Committee, and includes only statements from authentic printed reports or from those signed by responsible authorities in the respective communities.

RAILWAY ESTIMATES:

Design, Quantities and Costs. By F. Lavis. N. Y., McGraw-Hill Book Co., Inc.; Lond., Hill Publishing Co., Ltd., 1917. 12 + 608 pp., 99 illus., 9 x 6 in., cloth. $5.00.

A development of Chapter 8 of the author's "Railway Location, Surveys and Estimates", written to present in convenient form, data for the use of engineers called on to: (1) Report on the value of, or estimate the probable cost of, proposed railways, either before or after surveys have been made; (2) Estimate the value of existing

lines; (3) Design the general features of a proposed railway or modify the design of an existing line; (4) Determine the value or utility of such features of the general design of railways as affect their cost or value as transportation machines. It is not intended to cover the design of details of structures, but rather to present sufficient data for the determination of both quantities and costs to aid, not only in the preparation of estimates, but also in the determination of the general features of design.

SANITATION PRACTICALLY APPLIED.

By Harold Bacon Wood. N. Y., John Wiley & Sons, Inc.; Lond., Chapman & Hall, Ltd., 1917. 473 pp., 44 illus., 8 x 5 in., cloth. $3.00.

This book serves as a corollary to treatises on the theory of hygiene and to laboratory manuals. It is intended primarily for the health officer and the student of public health topics, but the practical directions given are useful to all. Illustrations are introduced as explanatory of the text. Contents: The Need for Public Health Work; Statistics; The Control of Communicable Disease; Child Welfare; School Hygiene; Pure Foods; Clean Milk; Water Supplies; Sewage Disposal; Hygiene of the Home and Factory; The Destruction of Insects which Transmit Disease; The Educational Movement; Equivalents in Weights and Measures.

RECENT PRACTICE IN THE SANITARY DRAINAGE OF BUILDINGS:

With Memoranda on the Cost of Plumbing Work. By Wm. Paul Gerhard. (Van Nostrand Science Series.) 2d ed., rev. and enl. N. Y., D. Van Nostrand Co., 1890. 175 pp., 6 x 4 in., boards. 50 cents.

HOUSE-DRAINAGE AND SANITARY PLUMBING.

By Wm. Paul Gerhard. 12th ed. (Van Nostrand Science Series.) N. Y., D. Van Nostrand Co., 1907. 231 pp., 6 pl., 6 x 4 in., boards. 50 cents. (Gift of the author and publisher.)

THE DISPOSAL OF HOUSEHOLD WASTES:

A Discussion of the Best Methods of Treatment of the Sewage of Farm-Houses, Isolated Country Houses, Suburban Dwellings, Houses in Villages and Smaller Towns, and of Larger Institutions, such as Hospitals, Asylums, Hotels, Prisons, Colleges, etc., and of the Modes of Removal and Disposal of Garbage, Ashes, and Other Solid House Refuse. By Wm. Paul Gerhard. 3d ed. N. Y., D. Van Nostrand Co., 1915. 195 pp., 4 illus., 6 x 4 in., boards. 50 cents.

STRENGTH OF MATERIALS.

By James E. Boyd. 2d ed., rev. and enl. N. Y., McGraw-Hill Book Co., Inc.; Lond., Hill Publishing Co., Ltd., 1917. 16 + 380 pp., 254 illus., 9 x 6 in., cloth. $3.00.

No radical changes have been made in the present edition of this book. A chapter on Curved Beams and Hooks has been added and part of a chapter on Theories of Failures. The most important addition is the method of Area Moments for deriving the equations of the elastic line of beam. The book has been designed to be used in connection with either "Cambria Steel" or "Carnegie Pocket Companion."

FARM CONCRETE.

By K. J. T. Ekblaw. N. Y., The Macmillan Co., 1917. 11 + 295 pp., 77 illus., 16 pl., 8 x 5 in., cloth. $1.60.

A simple, non-technical, yet reasonably comprehensive, account of the methods of using concrete for farm buildings, fences, drains, bridges, etc. Intended primarily for those without experience in the use of concrete.

PAINT RESEARCHES AND THEIR PRACTICAL APPLICATION.

By Henry A. Gardner. Washington, D. C. (privately printed), 1917. 384 pp., 155 illus., 9 x 6 in., cloth. $5.00.

This volume is a summary of the author's investigations conducted at the Institute of Industrial Research for the Educational Bureau of the Paint Manufacturers' Association of the United States. Contents: The Growth of the Prepared Paint Industry and Its Relation to the Work of the Painter; The White Pigment Industry; Physical Characteristics of Pigments and Paints; Tests of Lithopone; Washington Paint Oil Tests; Paint Protection for Portland-Cement Surfaces; Paints to Prevent Electrolysis in Concrete Structures; Paints for Metal; Marine Paints; Arlington Paint Tests; Observations on Painted Lumber; Impregnated Panel Tests; Fire Retardent Paints for Shingles and Other Wooden Structures; The Composition of Paint Vapors; The Toxic and Antiseptic Properties of Paints; The Light-Reflecting Values of White and Colored Paints; Formation and Inhibition of Mildew in Paints; Fungi on Painted Surfaces; Changes Occurring in Oils and Paste Paints, Due to Autohydrolysis of the Glycerides; The Effect of Pigments upon the Constants of Linseed Oil; Storage Changes in Vegetable and Animal Oils; Paint Driers and Their Application; Miscellaneous Oil Investigations; The Application of Paints and Finishes to Wood.

AMERICAN HYDROELECTRIC PRACTICE:

A Compilation of Useful Data and Information on the Design, Construction, and Operation of Hydro-electric Systems from the Penstocks to Distribution Lines. By William T. Taylor and Daniel H. Braymer. N. Y., McGraw-Hill Book Co., Inc.; Lond., Hill Publishing Co., Ltd., 1917. 14 + 439 pp., 258 illus., 9 x 6 in., cloth. $5.00.

Not a textbook on the fundamentals underlying the design and construction of the parts of a hydro-electric system, but a compilation of the practical and essential features of design, construction, and operation as used in many plants, interpreted and arranged for use by designers and engineers. Contents: General Survey of Water-Power Engineering; Low, Medium and Head Developments; Layout and Selection of Plant Equipment; Transmission Line Construction and Operation; Plant Line and Substation Costs; System Operation and Economics; Special Plant and Line Problems; Data, Reference Tables and System Diagrams.

MEMBERSHIP

(From August 3d to September 6th, 1917)

ADDITIONS

MEMBERS	Date of Membership.
BURKY, CHARLES ROGY. Chf. Engr., Twin Falls North Side Land & Water Co., Jerome, Idaho	May 15, 1917
CLEVELAND, MALCOLM COLBURN. Valuation Engr., L. V. R. R., 620 West 116th St., New York City	June 11, 1917
TROST, PAUL ANTHONY. Care, George F. Hardy, 309 Broadway, New York City	June 11, 1917

ASSOCIATE MEMBERS

BALDRY, WILLIAM EARNEST. City Engr., Topeka, Kans.	May 15, 1917
HANMER, FRANKLIN JOSEPH. Structural Engr., Lakeside Bridge & Steel Co., 861 Thirty-seventh St., Milwaukee, Wis.	June 11, 1917
HORMEL, ALFRED ALBERT. Res. Engr., George F. Hardy; Res., 100 Washington St., Brighton, Mass.	April 17, 1917
HOUSTON, ROBERT BRUCE. Junior Structural Engr., Interstate Commerce Comm., 601 Interstate Bldg., Kansas City, Mo.	April 17, 1917
JORDAN, LORING KENNETH. Anderson, Cal.	Mar. 13, 1917
MACINTOSH, PERCY HUGH MARSHALL. Box 2204, G. P. O., Sydney, New South Wales, Australia	Mar. 2, 1915
PRATT, EDMUND ADDISON. 1900 Land Title Bldg., Philadelphia, Pa.	May 15, 1917
SWAREN, JOHN WILLIAM. Vancouver Barracks, Vancouver, Wash.	June 11, 1917
THOM, NEIL, JR. 1315 Humboldt Bank Bldg., San Francisco, Cal. Jun.	Oct. 1, 1912
Assoc. M.	June 11, 1917

CHANGES OF ADDRESS

HONORARY MEMBERS

MILLS, HIRAM FRANCIS. Engr., Essex Co., 314 Main St., Hingham, Mass.

MEMBERS

BALDWIN, ERNEST HOWARD. Asst. Constr. Engr., Guggenheim Bros., Care, Braden Copper Co., Rancagua, Chile.

BENFIELD, BERNARD. Cons. Engr., 525 Rialto Bldg., San Francisco, Cal.

BIXBY, WILLIAM HERBERT. Brig.-Gen., U. S. A. (*Retired*); Pres., Mississippi River Comm., 428 Customhouse, St. Louis, Mo.

BLACKFORD, FRANCIS WEBSTER. Civ. and Min. Engr., P. O. Box 116, Miami, Okla.

BURWELL, ROBERT LEMMON. Specification Aide, U. S. Navy Dept., Bureau of Yards and Docks, 2011 Park Rd., N. W., Washington, D. C.

MEMBERS (*Continued*)

BYERS, CHARLES HOPKINS. Dist. Engr., Interstate Commerce Comm., 731 Wells Fargo Bldg., San Francisco, Cal.

CAMERON, HARRY FRANK. Maj., Engr. Officers' Reserve Corps, 301st Regiment, Camp Devens, Ayer Junction, Mass.

CHAMBERS, FRANK TAYLOR. Civ. Engr., U. S. N., Naval Operating Base, Hampton Roads, Va.

CRECELIUS, SAMUEL FORDER. Fort Leavenworth, Kans.

CROCKARD, FRANK HEARNE. Care, Nova Scotia Steel & Coal Co., Ltd., New Glasgow, N. S., Canada.

CROSBY, WALTER WILSON. Care, The Maryland Club, Baltimore, Md.

CUNNINGHAM, JOHN GEORGE LAWRENCE. Chf. Timekeeper, War Dept., 14th National Army Cantonment, Funston, Kans.

DAAE, HANS ANDREAS. Engr., Federal Constr. Co., The Call Bldg., San Francisco, Cal.

ELLIOTT, MALCOLM. Maj., Corps of Engrs., U. S. R., 309th Regiment, Engrs., Camp Taylor, Ky.

FOWLER, CHARLES EVAN. Cons. Engr., 3910 Woolworth Bldg., New York City.

FOX, JOHN ANGELL. Care, Wisner Estates, Inc., 38 South Dearborn St., Chicago, Ill.

GRANT, JOHN ROBERT. H. Q. R. E., 24th Div., B. E. F., France.

GRANT, KENNETH CROTHERS. Capt., Engrs., U. S. R., Constr. Div., Signal Corps, Union Station, Washington, D. C.

GRAY, ALEXANDER. Engr. in Chg., Dept. of Public Works, St. John Harbour, P. O. Box 1393, Saint John, N. B., Canada.

HARRIS, BORDEN BAKER. Cons. Engr., 35 Nassau St., New York City.

HEALY, JOHN FRANCIS. 520 Second St., Huntington, W. Va.

HUFF, CLYDE LESLIE. 18 Shepard Flats, Sioux City, Iowa.

LEAHY, MAURICE JOSEPH. 42 Bridge St., South Hadley Falls, Mass.

McCULLOH, ERNEST. Care, Charles L. Pillsbury Co., 813 Metropolitan Life Bldg., Minneapolis, Minn.

McFETRIDGE, WILLIAM SUTTON. Prin. Asst. Engr., B. & L. E. R. R. (Res. 10 North Main St.), Greenville, Pa.

McLURE, NORMAN ROOSEVELT. Strafford, Pa.

MATHEWSON, THOMAS KNIGHT. Care, Sr. Ing. Juan Tonkin, Parque Forestal 560, Santiago, Chile.

MÖNNICHE, TOLLEF BACHE. Lerida, Boquete, Panama.

NAGLE, JAMES C. Prof. of Civ. Eng., and Dean of Eng., Agricultural and Mech. Coll., College Station, Tex.

NORTH, ARTHUR TAPPAN. Contr. Engr., 1737 East 72d St., Chicago, Ill.

OAKES, JOHN CALVIN. Col., Engrs., National Army, Care, The Adjutant General, Washington, D. C.

O'HEARN, JOHN LYNCH. 738 Wilson Bldg., Dallas, Tex.

ORNELLAS, CHARLES EVARISTE D'. Ingénieur des Arts et Manufactures, 48 Avenue Malakoff, Paris, XVI, France.

MEMBERS (*Continued*)

PHARR, HARRY NELSON. 1085 Poplar Ave., Memphis, Tenn.

PILLSBURY, GEORGE BIGELOW. Col., Corps of Engrs., U. S. A., Camp Kearney, Linda Vista, Cal.

PRATT, ARTHUR HENRY. Capt., Engr. Officers' Training Camp, American Univ., Massachusetts and Nebraska Aves., Washington, D. C.

PUGH, MARSHALL ROGERS. 230 Poplar Ave., Wayne, Pa.

RIPLEY, HENRY CLAY. Cons. Engr., 480 Virginia Park, Detroit, Mich.

ROWELL, GEORGE FREEMAN. Res. Engr., Day & Zimmermann, 611 Chestnut St., Philadelphia, Pa.

SHAW, ARTHUR MONROE. Maj., Engr. O. R. C., Const. Quartermaster, Camp Beauregard, Alexandria, La.

SIBERT, WILLIAM LUTHER. Maj.-Gen., First Regular Div., Am. Expeditionary Forces.

SNOW, JONATHAN PARKER. Cons. Engr., 18 Tremont St., Room 1027, Boston, Mass.

STANFORD, HOMER REED. Civ. Engr., U. S. N., Navy Yard, Boston, Mass.

STRUCKMANN, HOLGER. Pres. and Gen. Mgr., Cuban Portland Cement Co.; Vice-Pres. and Gen. Mgr., Knickerbocker Portland Cement Co., 30 East 42d St., New York City.

TAYLOR, EDWIN ALEXANDER. Care, The Denver Union Water Co., Denver, Colo.

VAN NORDEN, ERNEST MAITLAND. Civ. Engr., New York Edison Co., 130 East 15th St., New York City (Res., 1530 East 14th St., Brooklyn, N. Y.).

WALKER, MERIWETHER LEWIS. Lt.-Col., Corps of Engrs., U. S. A., Care, 116th Engrs., Camp Greene, Charlotte, N. C.

WALLACE, JOHN FINDLEY. (*Past-President.*) Chairman, Westinghouse, Church, Kerr & Co.; Chairman, Ry. Terminal Comm. of Chicago, 390 West End Ave., New York City.

WEBB, DE WITT CLINTON. Civ. Engr., U. S. N.; Public Works Officer, Navy Yard, Philadelphia, Pa.

WHEELER, WALTER SCOTT. Cons. Engr., 82 Park St., Dover, N. H.

YATES, CHARLES COLT. With U. S. Shipping Board, Custom House, New York City.

ASSOCIATE MEMBERS

ALDERMAN, ERNEST SAMUEL. No. 7, Touraine Bldg., Fort Worth, Tex.

ALDRICH, ELBERT CHRISTOPHER. Granby, Mass.

ALFORD, WILLIAM VALORUS. Chf. Engr., Am. Vanadium Co., Garrettsville, Ohio.

ALLEN, MAYNARD EDWARD. Vice-Pres. and Treas., Bailey & Allen Co., 122 South Michigan Boulevard, Chicago, Ill.

ALSBERG, JULIUS. 5438 Ellis Ave., Chicago, Ill.

ARDERY, EDWARD DAHL. Maj., Corps of Engrs., U. S. A., 15 Custom House, Norfolk, Va.

ASSOCIATE MEMBERS (*Continued*)

AUSTILL, HURIEOSCO. Capt., Engr. Officers' Reserve Corps, American Univ., Washington, D. C.

AUSTIN, FRANK WILLIS. With Mideon Eng. Co., Chanute, Kans.

BAILEY, LEWIS PENN. Capt., 304th Regiment, Camp Meade, Admiral, Md.

BATTIE, HERBERT SCANDLIN. With Tennessee Coal, Iron & R. R. Co.; 1st Lieut., U. S. R., Eng. Div., 2025 Fifth Ave., Birmingham, Ala.

BECKJORD, JESSEE GARNET. Honolulu, Hawaii.

BELL, JOSEPH EDGAR. Care, Montana Power Co., 319 Electric Bldg., Butte, Mont.

BROWN, ROBERT HUSE. 21 West 127th St., New York City.

BUSSE, FRANZ AUGUST. Care, L. W. Koerner, R. R. No. 1, Metamora, Ind.

BUXTON, EDWIN WALKER. Eng. Dept., K. C. So. Ry., 215 East 30th St., Kansas City, Mo.

CANAGA, GORDON BYRON. Scio, Ohio.

CAREY, WILLIAM NELSON. Capt., 313th Engrs., U. S. A., Camp Dodge, Iowa.

CATLIN, HAROLD BURD. Res. Engr., Hammond Steel Co., Syracuse, N. Y.

CHAPIN, CHARLES WALTER. Hutchinson, Kans.

CHAPPEL, MILFORD EDWARD. Care, Norfolk County Farms, Inc., Wallacetown, Va.

CHESLEY, FRANK EPHRAIM. Care, Bldg. and Maintenance Dept., Gen. Elec. Co., Erie, Pa.

CHRISTENSEN, GEORGE ANDREW. Capt., Q. M. U. S. R., Camp Kearny, San Diego, Cal.

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DEATHS

ARNOLD, WILLIAM HARRY. Elected Member, May 1st, 1907; died, August 13th, 1917.

CRANDALL, CHARLES LEE. Elected Junior, June 7th, 1876; Member, October 5th, 1892; died August 25th, 1917.

Total Membership of the Society, September 6th, 1917, 8 405.

MONTHLY LIST OF RECENT ENGINEERING ARTICLES OF INTEREST

(July 2d to August 1st, 1917)

NOTE.—*This list is published for the purpose of placing before the members of this Society, the titles of current engineering articles, which can be referred to in any available engineering library, or can be procured by addressing the publication directly, the address and price being given wherever possible.*

LIST OF PUBLICATIONS

In the subjoined list of articles, references are given by the number prefixed to each journal in this list:

(2) *Proceedings*, Engrs. Club of Phila., Philadelphia, Pa.
(3) *Journal*, Franklin Inst., Philadelphia, Pa., 50c.
(4) *Journal*, Western Soc. of Engrs., Chicago, Ill., 50c.
(5) *Transactions*, Can. Soc. C. E., Montreal, Que., Canada.
(7) *Gesundheits Ingenieur*, München, Germany.
(8) *Stevens Indicator*, Hoboken, N. J., 50c.
(9) *Industrial Management*, New York City, 25c.
(11) *Engineering* (London), W. H. Wiley, 432 Fourth Ave., New York City, 25c.
(12) *The Engineer* (London), International News Co., New York City, 35c.
(13) *Engineering News-Record*, New York City, 15c.
(15) *Railway Age Gazette*, New York City, 15c.
(16) *Engineering and Mining Journal*, New York City, 15c.
(17) *Electric Railway Journal*, New York City, 10c.
(18) *Railway Review*, Chicago, Ill., 15c.
(19) *Scientific American Supplement*, New York City, 10c.
(20) *Iron Age*, New York City, 20c.
(21) *Railway Engineer*, London, England, 1s. 2d.
(22) *Iron and Coal Trades Review*, London, England, 6d.
(23) *Railway Gazette*, London, England, 6d.
(24) *American Gas Engineering Journal*, New York City, 10c.
(25) *Railway Mechanical Engineer*, New York City, 20c.
(26) *Electrical Review*, London, England, 4d.
(27) *Electrical World*, New York City, 10c.
(28) *Journal*, New England Water-Works Assoc., Boston, Mass., $1.
(29) *Journal*, Royal Society of Arts, London, England, 6d.
(30) *Annales des Travaux Publics de Belgique*, Brussels, Belgium, 4 fr.
(31) *Annales* de l'Assoc. des Ing. Sortis des Ecoles Spéciales de Gand, Brussels, Belgium, 4 fr.
(32) *Mémoires et Compte Rendu des Travaux*, Soc. Ing. Civ. de France, Paris, France.
(33) *Le Génie Civil*, Paris, France, 1 fr.
(34) *Portefeuille Economiques des Machines*, Paris, France.
(35) *Nouvelles Annales de la Construction*, Paris, France.
(36) *Cornell Civil Engineer*, Ithaca, N. Y.
(37) *Revue de Mécanique*, Paris, France.
(38) *Revue Générale des Chemins de Fer et des Tramways*, Paris, France.
(39) *Technisches Gemeindeblatt*, Berlin, Germany, 0, 70m.
(40) *Zentralblatt der Bauverwaltung*, Berlin, Germany, 60 pfg.
(41) *Electrotechnische Zeitschrift*, Berlin, Germany.
(42) *Proceedings*, Am. Inst. Elec. Engrs., New York City, $1.
(43) *Annales des Ponts et Chaussées*, Paris, France.
(44) *Journal*, Military Service Institution, Governors Island, New York Harbor, 50c.
(45) *Coal Age*, New York City, 10c.
(46) *Scientific American*, New York City, 15c.
(47) *Mechanical Engineer*, Manchester, England, 3d.
(48) *Zeitschrift*, Verein Deutscher Ingenieure, Berlin, Germany, 1, 60m.
(49) *Zeitschrift für Bauwesen*, Berlin, Germany.
(50) *Stahl und Eisen*, Düsseldorf, Germany.
(51) *Deutsche Bauzeitung*, Berlin, Germany.
(52) *Rigasche Industrie-Zeitung*, Riga, Russia, 25 kop.
(53) *Zeitschrift*, Oesterreichischer Ingenieur und Architekten Vereines, Vienna, Austria, 70h.
(54) *Transactions*, Am. Soc. C. E., New York City, $12.
(55) *Transactions*, Am. Soc. M. E., New York City, $10.
(56) *Transactions*, Am. Inst. Min. Engrs., New York City, $6.
(57) *Colliery Guardian*, London, England, 5d.
(58) *Proceedings*, Engrs.' Soc. W. Pa., 2511 Oliver Bldg., Pittsburgh, Pa., 50c.

(59) *Proceedings,* American Water-Works Assoc., Troy, N. Y.
(60) *Municipal Engineering,* Indianapolis, Ind., 25c.
(61) *Proceedings,* Western Railway Club, 225 Dearborn St. Chicago, Ill., 25c.
(62) *American Drop Forger,* Thaw Bldg., Pittsburgh, Pa., 10c.
(63) *Minutes of Proceedings,* Inst. C. E., London, England.
(64) *Power,* New York City, 5c.
(65) *Official Proceedings,* New York Railroad Club, Brooklyn, N. Y., 15c.
(66) *Gas Journal,* London, England, 6d.
(67) *Cement and Engineering News,* Chicago, Ill., 25c.
(69) *Der Eisenbau,* Leipzig, Germany.
(71) *Journal,* Iron and Steel Inst., London, England.
(71*a*) *Carnegie Scholarship Memoirs,* Iron and Steel Inst., London, England.
(72) *American Machinist,* New York City, 15c.
(73) *Electrician,* London, England, 18c.
(74) *Transactions,* Inst. of Min. and Metal., London, England.
(75) *Proceedings,* Inst. of Mech. Engrs., London, England.
(76) *Brick,* Chicago, Ill., 20c.
(77) *Journal,* Inst. Elec. Engrs., London, England, 5s.
(78) *Beton und Eisen,* Vienna, Austria, 1, 50m.
(79) *Forscherarbeiten,* Vienna, Austria.
(80) *Tonindustrie Zeitung,* Berlin, Germany.
(81) *Zeitschrift für Architektur und Ingenieurwesen,* Wiesbaden, Germany.
(83) *Gas Age,* New York City, 15c.
(84) *Le Ciment,* Paris, France.
(85) *Proceedings,* Am. Ry. Eng. Assoc., Chicago, Ill.
(86) *Engineering-Contracting,* Chicago, Ill., 10c.
(87) *Railway Engineering and Maintenance of Way,* Chicago, Ill., 10c.
(88) *Bulletin* of the International Ry. Congress Assoc., Brussels, Belgium.
(89) *Proceedings,* Am. Soc. for Testing Materials, Philadelphia, Pa., $5.
(90) *Transactions,* Inst. of Naval Archts., London, England.
(91) *Transactions,* Soc. Naval Archts. and Marine Engrs., New York City.
(92) *Bulletin,* Soc. d'Encouragement pour l'Industrie Nationale, Paris, France.
(93) *Revue de Métallurgie,* Paris, France, 4 fr. 50.
(95) *International Marine Engineering,* New York City, 20c.
(96) *Canadian Engineer,* Toronto, Ont., Canada, 10c.
(98) *Journal,* Engrs. Soc. Pa., Harrisburg, Pa., 30c.
(99) *Proceedings,* Am. Soc. of Municipal Improvements, New York City, $2.
(100) *Professional Memoirs,* Corps of Engrs., U. S. A., Washington, D. C., 50c.
(101) *Metal Worker,* New York City, 10c.
(102) *Organ für die Fortschritte des Eisenbahnwesens,* Wiesbaden, Germany.
(103) *Mining and Scientific Press,* San Francisco, Cal., 10c.
(104) *The Surveyor and Municipal and County Engineer,* London, England, 6d.
(105) *Metallurgical and Chemical Engineering,* New York City, 25c.
(106) *Transactions,* Inst. of Min. Engrs., London, England, 6s.
(107) *Schweizerische Bauzeitung,* Zürich, Switzerland.
(108) *Southern Machinery,* Atlanta, Ga., 10c.
(109) *Journal,* Boston Soc. C. E., Boston, Mass., 50c.
(110) *Journal,* Am. Concrete Inst., Philadelphia, Pa., 50c.
(111) *Journal of Electricity, Power and Gas,* San Francisco, Cal., 25c.
(112) *Internationale Zeitschrift für Wasser-Versorgung,* Leipzig, Germany.
(113) *Proceedings,* Am. Wood Preservers' Assoc., Baltimore, Md.
(114) *Journal,* Institution of Municipal and County Engineers, London, England, 1s. 6d.
(115) *Journal,* Engrs.' Club of St. Louis, St. Louis, Mo., 35c.
(116) *Blast Furnace and Steel Plant,* Pittsburgh, Pa., 15c.

LIST OF ARTICLES

Bridges.

Design and Construction Details of a Long Concrete Arch Bridge.* C. D. Hale. (67) June.
Reconstruction of Union Pacific Railroad Bridge at Omaha.* (11) June 8.
Three-Hinged Arch Highway Bridge Built of Timber.* F. W. Haselwood. (13) June 21.
Cost-Keeping System for Small Bridge and Culvert Jobs. (86) June 27.
Bascule Bridge at Syracuse Typical of New York Canal Crossings.* Arthur G. Hayden. (13) June 28.
A Viaduct Built by Maintenance Forces.* (87) July.
Bascule Bridge Across Cataraqui River; Details of Construction and Method of Operation of 160-Ft. Span Bascule Bridge at Kingston, Ont.—Precautionary Means Taken to Insure Counterweight Would Exactly Balance the Moving Leaf—How Weight Adjustment Was Determined.* R. K. Palmer. (96) July 5.

* Illustrated.

Bridges—(Continued).

Bids for Concrete Arch Viaduct Less Than for Steel Design.* H. G. Perring (13) July 12.
Standard Bridge Floors of Concrete Slabs on Steel Beams.* William Snaith. (13) July 12.
Locomotive Cranes Erect Cantilever Bridge Over Ohio River.* J. L. DeVou. (13) July 19.
Monumental Portal Completed at Approach of Manhattan Bridge.* (13) July 19.
Wide Roadway of Cleveland Bridge Concreted Cheaply.* (13) July 19.
Suspension Bridges in New Zealand. (12) July 20.
Bascule Bridge is Erected as a Cantilever.* W. J. Howard. (13) July 26.
Saving a Railway Bridge Built 63 Years Ago.* (96) July 26.
Cold Bituminous Mixture for Wood Floor Highway Bridges.* B. H. Piepmeier. (86) Aug. 1.

Electrical.

High Tension Transmission Lines, Poles and Towers.* Lesslie R. Thomson. (5) Oct.–Dec., 1916.
The Making of Rates After Valuation. William J. Norton. (4) May.
The Nature of the Power Requirements of the Electrochemical Industry. F. A. Lidbury. (4) May.
Dielectric Losses in Insulating Materials.* C. E. Skinner. (3) June.
Economical Combination of Water Power and Steam Plants and a Convenient Method of Solution.* H. St. Clair Putnam. (42) June; (64) July 24.
Electrolysis—Troubles Caused Thereby and Remedies Which may be Applied.* Albert F. Ganz. (28) June.
The Fundamentals of Successful High-Tension Cable Joints. D. W. Roper. (42) June.
Illumination of the Panama-Pacific International Exposition.* W. D'A. Ryan. (42) June.
The Influence of Dielectric Losses on the Rating of High-Tension Underground Cables.* A. F. Bang and H. C. Louis. (42) June.
Insulation Characteristics of High-Voltage Cables.* W. S. Clark and G. B. Shanklin. (42) June.
Present Practice in the Design and Manufacture of High-Tension Insulators.* A. O. Austin. (42) June.
Note on the Testing of Magneto Magnets.* J. D. Morgan. (11) June 15.
Modern Dynamo Electric Machinery.* Alexander Gray. (3) July.
Power House Economics.* V. T. Kropidlowski. (25) July.
Possibilities of Industrial Electric Heating.* (Discussion held at Pacific Coast Section of N. E. L. A.) (111) July 1.
Bell Transformers: Design and Tariffs.* E. Wirz. (73) July 6.
Electric Vehicles in Municipal Service. S. L. Pearce. (104) July 6.
New Concentric Standard Dynamometer Wattmeter for Heavy Currents: and Concentric Non-Inductive Standards of Low Resistance.* A. E. Moore. (73) July 6.
Power Station Centralization and Inter-Linking. John P. C. Kivlen. (Paper read before the Assoc. of Min. Elec. Engrs.) (22) July 6.
Foundry and Machine Shop Lighting.* C. E. Clewell. (27) July 7.
Underwriters Laboratories' New Testing Station.* Edgar P. Slack. (27) July 7.
Collinsville Plant Remodeled.* Thomas Wilson. (64) July 10.
Practical Talks on Controllers—A.-C. Contactor Switches.* (64) July 10.
Engineering Precautions in Radio Installations. Robert H. Marriott. (Abstract of paper read before the Inst. of Radio Engrs.) (73) July 13.
New Reversing Drive for Tin-Bar Mill at the Monmouthshire Steel and Tinplate Company's Works.* (22) July 13.
Phase Lag in Current Transformers.* A. J. Makower and A. Wust. (73) July 13.
Graphical Determination of Alternator Regulation.* A. S. Blatterman. (27) July 14.
Hints About Mill Lighting.* A. L. Powell. (16) July 14.
Illumination of Business Office Spaces.* C. E. Clewell. (27) July 14.
Methods of Making Temporary Motor Repairs.* H. L. Hayes. (27) July 14.
Bearing Currents in a Rotary Converter.* L. J. Corbett. (111) July 15.
New Power Station at Albany.* Charles H. Bromley. (64) July 17.
Depreciation of Electrical Machinery. William H. Easton. (72) July 19.
The Characteristics of Iron in High Frequency Magnetic Fields.* Ralph Bown. (73) July 20.
Leads for Electric Furnaces.* Arvid Lindstrom. (26) July 20.
The Prospects of the Accumulator. (73) July 20.
Standard Methods of Testing Moulded Insulation. (73) July 20.
Westinghouse Turbo-Alternators.* (26) July 20.
Analysis of Design-Constant Curves.* Theo. Schou. (27) July 21.
Comparison of Dissimilar Transformer Ratings. E. Hagenlocher. (27) July 21.
Illuminating Iron and Steel Mills.* C. E. Clewell. (27) July 21.
The Improvement of the Outdoor Substation.* M. M. Samuels. (27) July 21.

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Capacities of Transformers for Induction Motors.* G. P. Roux. (64) July 24.
Electric Controllers for Direct-Current Industrial Motors.* A. G. Popcke. (72) July 26.
Electric Cooking in the Factory.* (26) July 27.
Electricity Supply Costs.* Frank H. Whysall. (73) July 27.
Electrolysers and Their Management.* Ernest Reuss. (26) July 27.
Switch Which Prevents Failure of Signal Current.* H. K. Le Sure. (17) July 28.
The Unipolar Dynamo and Its Future.* Louis B. Lawrence. (19) July 28.
Ashley Street Station Remodelled.* John Hunter. (64) July 31.
Reconnecting Induction Motors—Fractional-Pitch Windings.* A. M. Dudley. (64) July 31.

Marine.

An Analysis of Model Screw Propeller Experiments.* S. Motora. (11) June 8.
The German High Sea Fleet and Its Bases.* (12) Serial beginning June 15.
The Stability of Submarine Boats While Trimming.* S. Motohara. (Abstract of paper read before the Japanese Soc. of Naval Archts.) (11) June 15.
The Science of Naval Architecture. David Watson Taylor. (19) June 23; (3) July.
The Submarine Boat: Its Inception and Development. John P. Holland, Jr. (19) June 30.
The Sidehaul Railway Dry Dock at West Memphis, Arkansas.* Clarke S. Smith. (100) July–Aug.
Housing the Navy Ashore.* (13) July 5.
The Real Problem of the Submarine.* (72) July 12.
Reinforced Concrete for Ships. (96) July 19.
Reinforced Concrete Ships.* J. L. Weller. (96) July 26.
American Standard Wooden Steamship.* (11) July 27.

Mechanical.

Cotton Rope for Power Transmission.* J. Melville Alison. (58) June.
Development of the Pickling Process: Consideration of Various Methods of Pickling—Conveying Machinery Comparisons—Data Concerning Per Cent. Acid and Time of Pickling for Various Materials and Methods. J. H. Snyder. (Abstract of paper presented at the Fourth Annual Convention of the Am. Drop Forge Assoc.) (62) June.
Electric Welding on the Rock Island.* E. Wanamaker. (25) June.
The Elements of Roll Pass Design: How Modifications in Existing Conditions Affect Supposedly Positive Rules—Fixing the Limitations on Possible Reduction per Pass—Action of Materials Under Compression. W. Trinks. (116) June.
Heat Treatment of Drop Forgings: Particular Attention to Automobile Parts—Methods of Determining Temperature of Steel During Heat Treatment—Checking Stock Distribution by Scleroscope Test—Rate of Heating. W. C. Peterson. (62) June.
Heat Treatment of Steel Forgings: Interesting Discussion Dealing with Failure in Heat Treatment of Forgings in English Works Practice—Advantages of Water Quenching—Recommendation That Research Work be more Clearly Applied to Shop Practice.* H. H. Ashdown. (Paper read before the Inst. of Mech. Engrs.) (62) June.
Locomotive Shovel Plates from Old Tires: Adjusting Rolling Mill Equipment to Handle Scrapped Tires. Problems Arising in the Process—Preparing Rolls for Heavy Duty Requirements.* W. S. Standiford. (116) June.
The Manufacture of Steel Castings: Rapid Developments of the Art of Forming Large Castings—Problems of Design and Treatment Briefly Sketched—Use of Acid and Basic Steels—Shrinkage, Porosity and Annealing Difficulties.* Robert P. Lamont. (Paper presented at the New York Meeting, Am. Iron and Steel Inst.) (116) June.
Power Tests of Machine Tools.* C. H. Crawford. (25) June.
Relative Methods of Forming Steel by Pressing, Hammering or Rolling.* John Lyman Cox. (Abstract of paper presented at New York Meeting, Am. Iron and Steel Inst.) (116) June.
Selection of Machine Tools.* Willard Doud. (25) June.
Standardizing of Finish on Forgings: Committee Suggested to Standardize Designs and Drawings and to Formulate Working Code for Forging Practice—Desirable Practice for Axles, Crank Shafts, Gear Blanks, etc. E. J. Frost. (Abstract from paper read before the Am. Drop Forge Assoc.) (62) June.
The Briquetting of Metal Swarf.* (12) June 8.
Control Gear for Auxiliary Motors for Rolling Mills.* (11) June 15.
Foundry Facings. H. Winterton. (Paper read before the Birmingham Branch of the British Foundrymen's Assoc.) (47) June 15.
Repetition Work and Machine Moulding. A. R. Bartlett. (Paper read before the London Branch of the British Foundrymen's Assoc.) (47) Serial beginning June 15.

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The Window-Glass Machine.* Robert Linton. (Paper read before Engrs.' Soc. of Western Pennsylvania.) (11) June 15.
The Spontaneous Ignition Temperatures of Liquid Fuels.* Harold Moore. (12) June 22.
Tar Oil Fuel and Diesel Engines. Geoffrey Porter. (Paper read before the Diesel Engine Users' Assoc.) (47) June 22.
Ball Mills for Grinding Ores.* (19) June 23.
The Gas Film Theory in Relation to Heat Conduction, and a Large Table of Heat Conductivities.* Halbert P. Gillette. (86) June 27.
Concentrated Potash a By-Product of Cement Mill. (13) June 28.
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Improvements in Vacuum Brake Apparatus.* (12) June 29.
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Welding with Application to Automobile Engineering.* Herbert L. Towns. (Abstract of paper read before Coventry Branch of Inst. of Automobile Engrs.) (11) June 29; (47) July 6.
Experiments with the Rotating Disk and Peripheral Alaries in Mechanical Flight.* Justin Kay Toles. (19) June 30.
The Elements of Roll Pass Design.* W. Trinks. (116) July.
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Power House Economics.* V. T. Kropidlowski. (25) July.
Roll Design for Shrapnel and Shell Bars.* W. S. Standiford. (116) July.
Structural Changes during Heat Treating: Describing How the Structural Changes Occur—A Critical Point in Heating Must be Obtained in Order to Get Results. James H. Herron. (Paper read before the Am. Drop Forge Assoc.) (62) July.
Welding by Electric Arc in Railway Shops in America. (21) July.
Progress in Aeronautics.* H. Bannerman-Phillips. (44) July-Aug.
Rational and Empirical Formulas for Steam Constants in Fuel Oil Practice.* Robert Sibley and Chas. H. Delany. (111) July 1.
The Manufacture of Linseed Oil. Glenn H. Pickard. (Paper read at Buffalo Meeting of Am. Inst. of Chemical Engrs.) (105) July 1.
The Recovery of Potash from Beet Sugar House Waste Liquors. H. E. Zitkowski. (Paper presented at Buffalo Meeting of Am. Inst. of Chemical Engrs.) (105) July 1.
Metallurgical Processes in the Foundry—Use of Ferromanganese, Ferrosilicon, Chills and Iron Molds.* Alexander E. Outerbridge, Jr. (Address delivered before Phil. Foundrymen's Assoc.) (105) July 1; (20) June 28.
Cutting Oils—Their Properties, Composition, Examination and Industrial Application. C. W. Copeland. (105) July 1.
Synthetic Production of Benzine and Toluene from Oil. Andrew Bender. (83) July 2.
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"Abrupt" Condensation of Naphthalene. (66) July 3.
Cost Data on Operation of Small Maintenance Trucks.* Frank T. Sheets. (86) July 4.
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Grinding Rolling-Mill Rolls.* Howard W. Dunbar. (72) July 5.
Inspectors and Inspection. A. L. Haas. (72) July 5.
Time Studies for Rate Setting on Gisholt Boring Mills. Dwight V. Merrick. (72) Serial beginning July 5.
The Conversion of Ford Cars into Commercial Vehicles.* (12) July 6.
Experiments in Low-Temperature Distillation.* (57) July 6.
Graphic Solutions of Some Compressed Air Calculations.* C. W. Crispell. (57) July 6.
The Latent Heat of Steam. Frank B. Aspinall. (12) July 6.
Galvanizing and Tinning Iron Wire. (19) July 7.
How Rope is Made.* Ernest Elva Weir. (46) July 7.
Recent Briquet-Plant Installations.* G. J. Mashek. (45) July 7.
The Complete Gasification of Coal. Alex. Naumann. (Abstract from *Chemiker Zeitung*.) (66) July 10.
Effect of Countershaft Speed Ratio on Power and Torque.* A. Lewis Jenkins. (72) July 12.
Keeping Machines on the Job.* Byron T. Hawley. (72) July 12.
Mechanical Equipment of a Hawaiian Sugar Plantation.* (72) July 12.
Panama Canal Boiler Shops at Balboa.* Frank A. Stanley. (72) July 12.
Condenser Tube Corrosion. William Ramsay. (11) July 13.
The Deterioration of Curtis Rateau Turbine Blading.* A. Fenwick. (Paper read before the South African Inst. of Engrs.) (12) July 13.
Combination Coal and Gas Firing.* Alex. W. Morgan. (27) July 14.

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Gas Displaces Electricity for Department Store Lighting.* H. S. Lewis. (24) July 14.
Handling the Coal from the Cars to the Stokers.* J. A. Longley. (27) July 14.
Standardizing Screens. (16) July 14.
The Use of Fuel Oil for Stationary Plants. Frederic Ewing. (Abstract of Discussion before A. I. E. E. and A. S. M. E.) (27) July 14.
A New Advance in Fuel Oil Economy.* C. D. Stewart. (111) July 15.
Compound Ammonia Compression.* H. J. MacIntire. (64) July 17.
Overcoming Cracking in Drying Clay Products. Otis L. Helfrich. (76) July 17.
Suggested Improvements in the Manufacture of Silica Brick.* C. E. Nesbitt and M. L. Bell. (76) July 17.
Alignment Chart for Milling Cutters.* A. Lewis Jenkins. (72) July 19.
The Maag Gear System.* Dexter S. Kimball. (72) July 19.
Stove Plate Foundry of a Modern Design.* (20) July 19.
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Physical Conditions Relative to the Heating of Regenerative Ovens.* D. Bagley. (22) July 20.
A System for Fuse Inspection. (12) July 20.
The Evolution of Artificial Dyestuffs: How American Workers are Reducing the Mazes of Coal-Tar Chemistry to Their Lowest Terms.* Thomas H. Norton. (46) July 21.
Tire Heating by Gas.* Gilbert C. Shadwell. (24) July 21.
Industrial Fuel from Gas-Works. E. W. Smith. (66) July 24.
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Coal Concreted from Dusts or Ashes. R. Goulburn Lovell. (From paper read before the Soc. of Archts.) (57) July 27.
The Down Draught Smokeless Boiler.* A. Bement. (57) July 27.
The Pulp and Paper Industry in Canada. O. F. Bryant. (29) July 27.
Some Experiences in Steel-Foundry Practice. T. Brown. (Abstract of paper read before the Sheffield Branch of the British Foundrymen's Assoc.) (47) July 27.
Economically Planned Coke Handling System at Denver Expedites Removal.* George Wehrle. (24) July 28.
The Selection of Gearing.* W. H. Phillips. (16) July 28.
Wire Drawing. A. T. Adam. (19) July 28.
Dempster-Toogood Continuous Vertical System.* (66) July 31.
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Proper Draft Conditions in Burning Kilns.* Arthur E. Williams. (76) July 31.
Refractory Brick—Their Properties and Constituents.* (76) July 31.
L'Amélioration du Fonctionnement Thermique et Organique des Moteurs à Explosion et à Combustion.* L. Pierre-Guédon. (33) July 7.
La Distillation de la Houille a Basse Température.* (33) July 17.

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Alumina as a Component of Furnace Slag: Characteristics Due to Alumina; Relations It Bears to Other Slag Components and the Practical Interpretation of Slag Peculiarities Which are Attributed to Alumina.* Wallace G. Imhoff. (116) June.
The Chemical Reactions of Iron Smelting. Walter Mathesius. (Abstract of paper read before New York meeting of Am. Iron and Steel Inst.) (116) June.
Metal Melting. H. M. Thornton. (Abstract of paper read before Inst. of Metals.) (11) Serial beginning June 15.
Wide Diversity of Uses of Gas-Fired Soft Metal Furnaces.* Gilbert C. Shadwell. (24) June 23.
A New Design of Gas-Fired Crucible Furnace.* (22) June 29.
Effect of Corrosion on the Ductility and Strength of Brass.* Paul D. Merica. (19) June 30.
Installation of Large Turbo-Generators: Specifications for Four Blast Furnace Plant.* Richard H. Rice. (Abstract of paper read before the Am. Iron and Steel Inst.) (116) July.
Sulphur as a Component of Furnace Slag.* Wallace G. Imhoff. (116) July.
Cobaltcrom, the New Steel Alloy. (11) July 6.
Electric Steel Furnaces, with Special Reference to the Greaves-Etchells Type.* E. Kilburn Scott. (22) July 6.
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Copper Smelting at Santa Lucia.* Francis Church Lincoln. (16) July 7.
Properties and Structure of Nickel Steel.* S. W. Parker. (20) July 12.
Ammonia Leaching of Calumet Tailings.* C. Harry Benedict. (16) July 14.
The Burro Mountain Concentrator.* F. C. Blickensderfer. (16) July 14.
Concentration at Humboldt, Arizona.* George M. Colvocoresses. (16) July 14.
Continuous Drum Filters.* Carl J. Trauerman. (16) July 14.
Drying Flotation Concentrates. H. H. Adams. (16) July 14.
Inhibitory Films in Flotation.* Will H. Coghill. (16) July 14.
Milling Wisconsin Lead-Zinc Ores.* Walter Schultz. (16) July 14.
Oil Concentration of Lead Carbonate at Darwin, California.* H. S. Rexworthy. (16) July 14.
Selective Lead-Zinc Flotation.* Luther L. Kirtley. (16) July 14.
Solution of Grinding Problems.* Sergio Bagnara. (16) July 14.
The Tom Reed Mill, Oatman, Ariz.* H. P. Flint. (16) July 14.
Sands Used in Metallurgical Practice. P. G. H. Boswell. (Abstract of paper read before the Soc. of Chemical Industry.) (57) July 20.
Milling Costs at the Inspiration and Bunker Hill Plants. (16) July 21.
Sulphatizing with Weak Sulphurous Gases.* George C. Westby. (16) July 21.
The United Verde Crushing Plant.* T. C. Roberts. (16) July 21.
War Tasks of the German Steel Industry.* (20) July 26.
An Electrical Method of Hardening Steel.* (12) July 27.
Flotation—The Butte & Superior Case. Walter A. Scott. (103) Serial beginning July 28.
Grain Size Measurements in Metals.* Zay Jeffries. (Paper read before the Faraday Soc.) (19) July 28.
The Heavy Shall Rise and the Light Shall Fall.* (46) July 28.
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Military Sanitation. W. P. Chamberlain. (109) June.
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Canadian Cantonments All Built by One Contractor. (13) June 28.
Handling 8-In. Shell Forgings.* M. E. Hoag. (72) June 28.
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The German Shelters on the Somme.* (Translated from *L'Illustration* by W. R. Livermore.) (122) July-Aug.
The Influence of the European War Upon the Art of Field Fortifications.* F. B. Wilby. (100) July-Aug.
Outline of a Plan for Effecting Universal Military Training in the United States. Stephen M. Foote. (44) July-Aug.
Forging Shells at Curtis Plant, St. Louis.* (20) July 5.
The Lewis Machine Gun.* (72) July 5.
Plans for Army's Big Training Camps Made Public.* (13) July 5.
Military Motor Truck Design. H. D. Church. (19) July 7.
The Military Rifle: Many Delicate Operations Required to Produce a Perfect Weapon. (19) July 7.
Re-introducing the Concrete-and-Steel Fortification.* (46) July 7.
Limits and Tolerances for the Manufacture of Munitions. (72) July 12.
United States Munitions: The Springfield Model 1903 Service Rifle: Operations on the Hand Guard.* (72) Serial beginning July 12.
Sanitation in Camps "Somewhere in France": Methods Developed During the Present War and Used by the British in Their Temporary Army Camps.* (101) July 13.
Automatic Pistols: Mechanism of a Recently Adopted Type of Military Arms.* H. Volta. (19) July 14.
Build Army Aviation Camp Near Detroit.* (13) July 19.
Forging Rifle Barrels.* Fred H. Colvin. (72) July 19.
Sighting Telescopes for Fire-Arms: Instruments that Aid in Precision in Long Distance Shooting.* Ernest Coustet. (19) July 21.
Construction and Structural Features of Cantonments for New York National Army.* (86) July 25.
Engineering in the Great European War.* Frank W. Skinner. (19) July 28.
Flat Site at Chillicothe Camp Demands Careful Drainage.* (13) Aug. 16.

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A Contribution to the Micro-Petrology of Coal.* George Hickling. (106) July.
Forms of Electric Power Best Suited for the Various Loads Encountered in the Operation of Bituminous Coal Mines. R. L. Kingsland. (42) July.
The Higher Training of Colliery Managers. G. L. Kerr. (106) July.

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Can Your Mining Methods be Improved? (76) July 3.
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Underground Signalling Bells.* L. Fokes. (22) July 6.
Efficiency of Coal Production. C. B. Officer. (45) July 7.
Growth of the Talc Industry.* Freeland Jewett. (16) July 7.
Longwall Mining in Northern Illinois. Edward T. Bent. (Abstract of address before Ill. Min. Inst.) (45) July 7.
The Scope and Value of State Mining Bureaus. E. K. Soper. (Abstract of address before Idaho Min. Assoc.) (16) July 7.
The Development of the Petroleum Industry in Assam. H. S. Maclean Jack. (29) July 13.
Rescue and Recovery Operations in Coal Mines. James W. Paul. (57) July 13.
Forestry as an Adjunct to Mining. J. Crow Taylor. (45) July 14.
Modern Coal Mining Plant at Arista, W. Va.* J. F. Morris. (45) July 14.
The Sugar Creek (Ohio) Mine Fire. (45) July 14.
A Two Stacker Gold Dredge.* (46) July 14.
Methods of Dealing with Gob Fires.* S. J. Bridges. (22) July 20.
Risk of Gas Ignition by Underground Magneto Telephones.* L. Fokes. (22) July 20.
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A Fourteen Hundred Foot Raise.* Charles H. Foreman. (16) July 21.
Handling Mine Water.* H. E. Cole. (45) July 21.
Mining Machinery as a Factor in the War.* (45) July 21.
New Method in Screening, Picking and Loading Bituminous Coal.* (45) July 21.
New Thin-Seam Mounting for Jackhammers.* H. L. Hicks. (45) July 21.
Observations on Modern Coal Washing.* G. H. Elmore. (45) July 21.
Prospecting an Iron Ore Property.* J. F. Kellock Brown. (16) July 21.
Sources of Sulphur in Oils. G. Sherburne Rogers. (103) July 21.
Survey of Inclines Without Auxiliaries.* A. J. Sale. (103) July 21.
Automatic Safety Mine-Car Cager.* N. L. Harmon. (45) July 28.
Drill Bits and Drill Steel for Metal Mining.* George H. Gilman. (16) July 28.
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Terry Turbines at Coal Mines.* (45) July 28.

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Some Suggestions for Improvements in the Engineering Profession. F. G. Jonah. (Paper read before the Engrs.' Club of St. Louis.) (115) May–June.
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Hammer Hour System of Overhead: Consideration of the Problem of Dividing the Forge Shop Overhead Among the Various Groups of Equipment in the Shop Itself—Tables Showing Non-Productive Labor, Purchase and Expense Items. R. T. Herdegen. (Abstract from paper read at the Fourth Annual Convention of the Am. Drop Forge Assoc.) (62) June.
Public-Service Regulation. Thomas W. D. Worthen. (28) June.
Intensive Production Establishment Charges and Selling Price.* (Paper read before the Exeter Chamber of Commerce.) (12) June 8.
Character the Most Essential Qualification for Success in Engineering. W. A. Cattell. (24) June 23.
British Decimal Coinage: A Choice of Unit. Harry Allcock. (22) June 29.
A Correct Financial Plan the Most Important Element in the Success of a Public Utility. G. E. Claflin. (24) June 30.
The Physical Basis of Color-Technology.* M. Luckiesh. (3) July.
Valuation of Public Service Property—Overhead Expense. A. S. B. Little. (83) July 2.
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An Ionization Manometer. O. E. Buckley. (19) July 7.
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The Action of Aluminium Chloride on Pure Aromatic Hydrocarbons.* Robert J. Moore and Gustav Egloff. (105) July 15.
Fundamental Hue Scale for Scientific Color Designation.* (105) July 15.
Notes on Public Utility Rates. C. E. Grunsky. (111) July 15.
The Possibilities of Developing an American Potash Industry. Richard K. Meade. (Paper read before the Am. Inst. of Chemical Engrs.) (105) July 15.
Surety Bonds Under Government Contracts. (86) July 18.
Knife-Edge Materials and Stresses.* H. L. Van Keuren. (Abstract of paper read before National Scale Men's Assoc.) (15) July 20.
Shall Great Britain and America Adopt the Metric System. Walter Renton Ingalls. (Paper read before the Inst. of Min. and Metallurgy.) (29) July 20.
Exploring the Realms of Science for Photographic Improvements.* (46) July 21.
Protecting Our Timber Resources: Using the Heliograph to Fight Forest Fires.* Arthur L. Dahl. (19) July 21.
Making Industrial Valuation for Tax Purposes. George E. Barrows. (13) July 26.
Against the Compulsory Adoption of the Metric System. H. Cunliffe. (From paper read before the Incorporated Soc. of Inspectors of Weights and Measures.) (104) July 27.
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Lighting in the Textile and Clothing Industry.* C. E. Clewell. (27) July 28.
True Efficiency in Engineering Requires a Thorough Knowledge of the Human Factor. John P. Frey. (24) July 28.

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City Managers: A New Opportunity for Engineers. (86) July 25.
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Centralized Production of Locomotive Repair Parts.* George Armstrong. (25) June.
Developments in Railway Shop Tools. (25) June.
Enter Roller Bearings for Steel Mill Cars: The Steel Industry Takes a Hand in the Rolling Stock Efficiency Game—Interesting Tests by Steel Companies on Train Resistance, Made by a Dynamometer Testing Device—Results of Findings.* P. B. Liebermann. (116) June.
Grinding and Milling Work (Pennsylvania Railroad Shops).* (25) June.
Locomotive Feed Water Heating.* (25) June; (47) July 20.
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Milling Machines in Railroad Shops.* Harvey De Witt Wolcomb. (25) June.
Santa Fe Double-Deck Stock Cars.* (25) June.
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Locomotive and Freight Car Capacities and Loads; Maintenance Costs of Freight Cars and Freight Locomotives per Ton-Mile; Cost of Fuel for Freight Locomotives per Ton-Mile, for Fiscal Year 1916. (18) June 23.
Pacific and Santa Fe Type Locomotives for the St. Louis-San Francisco Ry.* (18) June 23.
Dragline Eliminates Haulage Equipment on Railroad Work.* W. A. Pillans. (13) June 28.
Double-Tracking a Railroad Through a Mountain Canyon.* (46) June 30.
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Conversion of Simple Expansion to Compound Locomotives on French Railways.* (21) July.
Erie Steel Passenger Equipment for Main Line Service: Important Features are Light Weight and Strength of the Superstructure.* (25) July; (18) July 7.
Graphic Display of Individual Daily Fuel Records.* Hiram J. Slifer. (Abstract of paper presented to the International Ry. Fuel Assoc.) (25) July.
History of the Development of Rail on the Baltimore and Ohio Railroad.* Earl Stimson. (58) July.

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AMERICAN SOCIETY OF CIVIL ENGINEERS

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PAPERS AND DISCUSSIONS

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DETENTION RESERVOIRS WITH SPILLWAY OUTLETS AS AN AGENCY IN FLOOD CONTROL

By H. M. Chittenden, M. Am. Soc. C. E.

To be Presented October 17th, 1917.

1.—Synopsis.

The writer's attention was particularly drawn to a possible working relationship between reservoir and spillway, as hereinafter set forth, by his study of the detention reservoir system while acting as consulting engineer to the Miami Conservancy District on certain occasions during the 30 months prior to the adoption of the Official Plan for flood protection in that district. He became convinced that applications of the detention principle were by no means exhausted in the Miami system, elaborate and well-considered as it is, but that there is a wider scope to its utility than even that notable example would indicate. The purpose of this paper is to inquire how far, and by what means, the detention principle may be made to harmonize the conflicting conditions, in reservoir development, between flood control and storage for industrial or other use. It is generally conceded that flood control of itself will not justify a very wide reservoir development, and that such justification must be sought in storage for use. It is further generally recognized that flood control and storage for use are

Note.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and, when finally closed, the papers, with discussion in full, will be published in *Transactions*.

essentially antagonistic purposes which make impossible a reliable utilization of the same reservoir space for both. In spite of these adverse conditions, however, there is a sub-conscious feeling among engineers that two such important purposes ought to be brought into more harmonious relations. In line with this sentiment the writer will endeavor in this paper to indicate a method by which this most desirable end can be at least partly, if not wholly, attained.

In view of the widespread interest in the question of flood control which has developed during the past few years, it is hoped that members of the Society, and others as well, who share this interest, will contribute from their experience or study to a further elucidation of the subject. A consensus of well-considered opinions, by men professionally qualified, cannot fail to be of public value.

2.—Detention Reservoirs: Definition and Terminology.

A detention reservoir is one formed by a natural or artificial engorgement in the valley of a stream whereby the volume of water which may pass for a given depth in a given time is strictly limited. The surplus which cannot thus pass in time of heavy flow accumulates in the basin above the engorgement and flows out as the inflow decreases. The function is purely automatic, human control being wholly eliminated. The effect is to prolong the period in which an excessive run-off passes the point of engorgement and to reduce proportionally the rate of discharge in the valley immediately below.

The engorgement may be produced in a variety of ways, as by the contraction of a natural chasm, by the construction of a dam with one or more conduits through the base, or by the construction of a dam with a spillway at some level above the base. In the first two cases the accumulated excess flows out entirely after the flood is past and the basin above is left empty. In the third case there will remain a body of water in the basin after the spillway has performed its alloted function. Although the first two types will be referred to frequently in this paper, only the third will be given special consideration, as it is the type which seems to be adaptable in some degree to the compromise of conflict referred to in Section 1 of this paper.

A brief reference to the terminology of the subject may properly be made at this point. The terms "detention reservoir" or "detention

basin", "retarding basin", "restraining reservoir", "impeding reservoir", have been used in official discussions of the subject. There is one defect common to all these terms, except possibly the third, and that is this—they do not suggest the end for which the system is designed, but only the means of attaining that end. Reduction of flood discharge is the great desideratum, but there is no hint of this in three of the foregoing terms. The terms "reduction" or "restriction" would be more appropriate, but there is something about these words that does not appeal to the ear. The term "restraining" carries a suggestion of both end and means, is in itself an agreeable descriptive, and perhaps fills all the requirements better than any other term that could be selected. Common use, however, is the final arbiter of all questions of this sort, and, on this basis, "detention reservoir" has the lead, notwithstanding the great prominence given to "retarding basin" by its use in the Official Plan of the Miami Conservancy District. As between "basin" and "reservoir", the writer distinctly prefers the latter, except in those cases where the basin itself and not its content is specifically referred to. As the agencies herein discussed are in the most literal sense reservoirs, "detention reservoir" is definitely adopted in this paper.

The term "dry reservoir" appeals to the writer as a most happy descriptive of that particular type of detention reservoir in which the detained excess of run-off runs out soon after the crisis of the flood, leaving the basin above the engorgement empty or dry. The original detention reservoir formed by the *Digue de Pinay* in France more than two centuries ago was of this type, as are also to be all the units of the Miami system.

3.—The Spillway: Definition and Description.

In its ordinary use the spillway is a device for conveying, over or through the crest of a dam, the surplus inflow from above in such a way as not to endanger the integrity of the dam. Its purpose is negative rather than positive. It plays no active part in the dynamic functions of the dam, as a penstock and other appurtenances do, but simply stands guard to see that the dam shall not be wrecked by any inflow in excess of that which its purpose of utility requires.

The successful operation of the spillway requires that it satisfy three vital conditions. It must have sufficient capacity to prevent overflow of the dam, because such overflow is always dangerous, and,

in the case of earthen dams, fatal unless promptly checked. The spillway must have sufficient structural resistance to withstand the tremendous strain of deep overflow. Means must also be provided at the point where the overflow strikes the level of the stream below to neutralize the energy developed in its fall without danger of undermining the dam. Inadequate satisfaction of these conditions is more common than the Engineering Profession likes to admit, and Professor Mead very properly says* that "perhaps there has been no more frequent cause of the failure of dams than inadequate spillways."

Inasmuch as the spillway requires a certain depth of overflow to be effective, and as it sometimes happens that the resulting elevation of water surface above the dam may be objectionable, the escape of water in such cases may be accelerated by the use of sluiceways, movable gates, flash-boards, and similar devices, all of which are under direct control. These serve the primary purpose of the spillway in facilitating the escape of surplus water, but lack the automatic character which is a distinctive feature of the true spillway.

The writer has defined the spillway, in the common acceptation of the term, as a safety device pure and simple. In the following discussion he will treat of it under a somewhat broader conception—one based not at all, or at least only incidentally, on the negative consideration of safety, but on positive considerations of utility. It will fulfill a vital function in the purpose of the dam as an agency of flood control, and will be a direct means of extending the application of the detention principle to cases heretofore considered of doubtful feasibility.

4.—Elements of Control.

As a preliminary to this section, it may be observed that the combined function of dam and spillway which denotes the true detention principle is universally exemplified in Nature. In almost infinite combination its elements of control may be found in actual operation. Every lake or pond which has an outlet is a case in point. Fig. 1 exhibits in graphic form the effect of Lake Kachess, Washington, in reducing the flow of the Kachess River in the great flood of November, 1906.† It will be noted that, as the run-off from the water-shed

* "Water Power Engineering," edition of 1915, p. 612.

† For the data on which this diagram is based the writer is indebted to G. L. Parker, Assoc. M. Am. Soc. C. E., District Engineer for the State of Washington, Hydrographic Branch, U. S. Geological Survey.

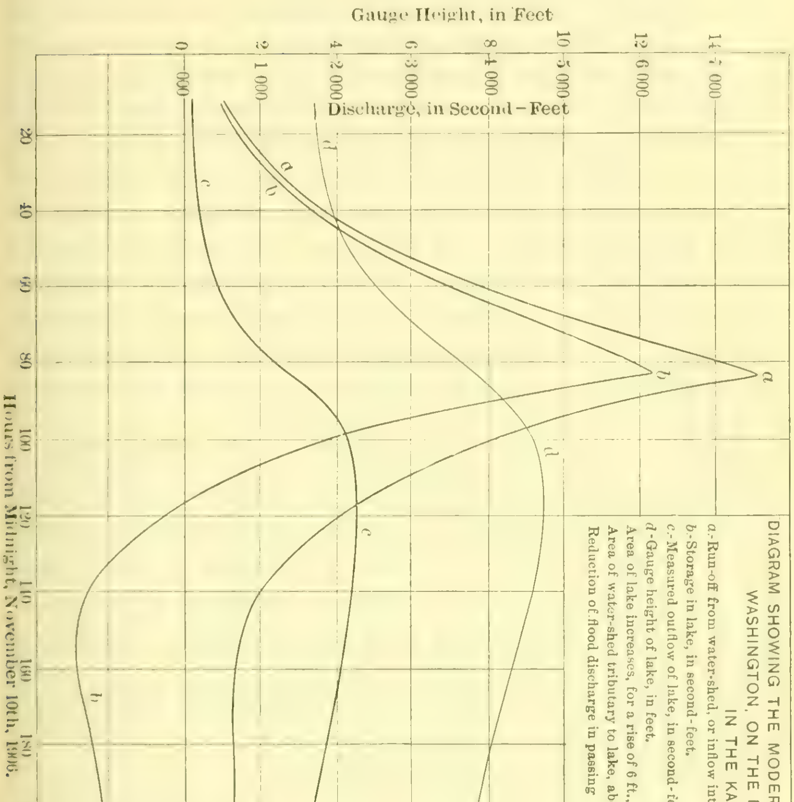
DIAGRAM SHOWING THE MODERATING EFFECT OF LAKE KACHESS, WASHINGTON, ON THE FLOOD OF NOVEMBER, 1906, IN THE KACHESS RIVER.
a.-Run-off from water-shed, or inflow into lake, in second-feet, Algebraic sum of b and c.
b.-Storage in lake, in second-feet.
c.-Measured outflow of lake, in second-feet.
d.-Gauge height of lake, in feet.
Area of lake increases, for a rise of 6 ft., from about 3 600 to about 3 840 acres.
Area of water-shed tributary to lake, about 40 000 acres.
Reduction of flood discharge in passing through lake, 70 per cent.
Gauge Height, in Feet
Discharge, in Second-Feet
Hours from Midnight, November 10th, 1906.
0
2
4
6
8
10
12
14
000
1 000
2 000
3 000
4 000
5 000
6 000
7 000
20
40
60
80
100
120
140
160
180
200
220
240
260
a
b
c
d

(inflow into the lake) begins to exceed the capacity of the outlet, the surplus accumulates in the reservoir, causing a rise in surface level and a consequent increase in outflow. This process continues until outflow balances inflow, at which point the curve of outflow, and the reservoir level, reach their maximum elevation, the curves of inflow and outflow cross each other, and the curve of storage passes through zero. The inflow curve always reaches its crest and is on the decline before it is overhauled by the rising outflow curve. The difference in time between the two crests varies greatly in different reservoirs for similar freshet conditions, and in the same reservoir for different freshet conditions. The net result, however, is in every case a temporary detention of a portion of the flood wave in the reservoir, a prolongation of the period of outflow, as compared with that of inflow, and a consequent reduction in the rate of discharge as the freshet passes through the reservoir.

The example of Lake Kachess is fairly typical, but Nature presents all degrees of control, from that which is practically complete to that which is practically negligible. Lake Superior stands at one extreme. So great is its area that the increment or decrement of outflow, compared with the corresponding increment or decrement of storage, for any rise or fall in the lake level, is quite insignificant. Variations of inflow are almost wholly compensated in storage, and the discharge of the outlet never varies by more than a small percentage from its mean value. The phenomena of high and low water, so characteristic of ordinary streams, are here entirely absent.

At the opposite pole from Lake Superior may be cited those natural widenings of channels occasionally found in some of our larger streams, and which have very slight storage capacity compared with the volume of water passing through them. The increment of storage in such cases is very small, as compared with the increment for the outflow of a given rise, and little regulative effect is experienced by a flood wave during its passage.

It is manifest that the moderating effect of detention reservoirs, whether natural or artificial, is the combined result of three co-operating factors, two of which are, or may be made, subject to artificial control, but the third is not. These are the storage capacity of the reservoir per unit depth at any level, the spillway capacity per unit

length, and the duration of the flood wave.* In perfectly general terms, this moderating effect increases with the area of the reservoir and diminishes as the length of the spillway, or the duration of the freshet, increases; but these variations in result never follow in exact ratio the variations in the factors which produce them. In some conditions the correspondence is close, in others it is widely divergent.

Assume first fixed conditions as to spillway length, and as to duration and volume of flood wave or inflow into the reservoir. The moderating effect of the reservoir would increase with an increase of area, but, as a rule, in slightly greater ratio. For example, with an area in one case twice as great as in another, the same quantity of storage would raise the larger surface half as much as it would the smaller, if the outflow in both cases was the same; but the outflow would not be the same. The flow over a spillway increases in faster ratio than the depth—generally much faster—and to cut off half the depth from the top would reduce the outflow by more than half. This is slightly compensated, however, in increased storage and elevation of surface due to restricted outflow; but, unless the spillway is relatively large, this effect would be small, and the reduction of outflow would remain more than half.

With fixed conditions as to area, volume, and duration of flood wave, the regulative effect will diminish with an increase in length of spillway, but in somewhat less ratio, because the greater outflow will diminish the height to which the reservoir surface will rise and consequently the outflow per unit length of spillway. Thus, if the spillway were doubled in length, the outflow, with fixed conditions as to the other factors, would be something less than doubled.

Much more difficult to follow than either of the foregoing cases is the effect of variations in the time factor, or the duration of inflow into the reservoir. It can only be stated, as a general proposition, that the regulative effect on the flood wave of given volume is greater as its duration is less; but there are so many qualifying conditions—as, for example, the relative size of spillway and the distribution of inflow during the freshet period—that no conclusion of general applicability seems possible.

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* In this discussion it is assumed that the capacity of a reservoir per unit depth at any level is the same as the area for that level; and that the capacity of the spillway for any depth varies directly with its length. Variations in these capacities, therefore, may be expressed in terms of variations in area and length, respectively.

Indeed, the whole problem of the inter-relation of these several factors seems to be too complicated to be expressed in any general formula which will embrace them all, and it seems destined to remain simply a problem in "particular cases"—the making of specific assumptions, and the determination of each group by itself. That is what would be done in practice anyway, but it would be a satisfaction, nevertheless, to have a general formula.

5.—Prevalence of the Reservoir Idea.

In passing from the general principles just discussed to their specific application, it may be stated at the outset that there is a deep-seated belief in the lay mind, and to a less extent in the mind of the expert, that in reservoirs is to be found the solution of the flood problem. As the writer stated some 20 years ago*: "To store the surplus water in the flood season and use it in the season of drought ought, apparently, to strike at the root of the whole difficulty. * * * Why so obvious a remedy has never yet been extensively applied," the writer at that time traced to a prohibitory disproportion between cost and resulting benefits. Although this is true, as a broad generalization, it will be more useful to the student of these questions to give some of the specific details on which the generalization rests. They may be summarized briefly as follows:

Deficiency of Sites.—This sometimes arises from an actual absence of physical sites, but more often from the lack of those which are economically feasible. Along the main valleys of the lower Ohio, Missouri, and Mississippi there are no sites whatever into which the main streams could be poured, except possibly to a small extent the overflow basins along the Mississippi. On the other hand, on almost any of the upper tributaries may be found physically practicable sites of sufficient capacity, if properly developed, to insure effective flood control. To-day, however, many of these sites are crossed by important railway systems which cannot be well re-located, or are occupied by villages and cities, rich and highly developed farms, mineral properties, etc., and, of course, almost all are traversed by public highways. It thus results that the occupancy of such sites for reservoir purposes, though physically practicable, is often economically prohibitory.

* "Reservoir Sites in Wyoming and Colorado", House Doc. 141, 55th Cong., 2d Sess., p. 32.

As a flood control measure, pure and simple, a reservoir may be only a trade-off, with the balance sheet against it. To quote again from the writer's early reservoir report (p. 46):

"Floods are only *occasional* calamities at worst. Probably on the majority of streams destructive floods do not occur, on the average, oftener than once in five years. Every reservoir built for the purposes of flood protection alone would mean the dedication of so much land to a condition of permanent overflow in order that three or four times as much might be redeemed from occasional overflow. One acre permanently inundated to rescue three or four acres from inundation of a few weeks once in three or four years, and this at a great cost, could not be considered a wise proceeding, no matter how practicable it might be from engineering considerations alone."

Of course, there are other considerations of a different character than mere overflow of bottom-lands which weigh on the other side of the balance. Moreover, the "dry" reservoir quite obviates the special objection of permanent overflow.

As a rule, reservoirs are practicable only on comparatively small streams; almost never, as already pointed out, on trunk streams of great magnitude. As floods in the trunk streams are invariably the result of tributary accessions, and as the magnitude of such floods is enhanced by the synchronous arrival of tributary flood peaks, any disturbance of the rate or duration of tributary flow might have an adverse effect on the combination. Ordinarily, the reduction of a peak on a regulated tributary would more than offset such occasional adverse effect. Nevertheless, an aggravation of natural conditions is a possibility. This matter will receive further consideration later.

The effect of reduction of a flood peak on a small tributary diminishes rapidly with the distance down stream, as other tributaries come in, and amounts to little or nothing when it reaches the lower course of a large river. Even so extensive a system as that of the upper Mississippi produces no appreciable effect below the mouth of the Missouri.

Economic considerations, as already indicated, are on the whole adverse to any general development of the reservoir system for flood control alone. Industrial or other use is generally necessary to justify the cost. Flood control and industrial use, however, conflict with each other to a certain extent, as hinted in Section 1 of this paper, and as will be more fully explained in Section 6.

That the drawbacks just mentioned are not at all imaginary, but are very real and of wide application, is abundantly proved by the paucity of examples of reservoirs built for the primary purpose of flood control. There has developed in the Engineering Profession a feeling somewhat akin to pessimism on the subject, and this has been aggravated by the ill-considered advocacy of certain visionary projects which has cast doubt, if not ridicule, on the fundamental principle itself; but there is clearly a middle ground. If reservoirs alone can never solve the flood problem on all our streams—and they certainly cannot—they may probably do more than most of us in recent years have believed to be possible. Omitting those rare and extraordinary sites, relatively few in number, where dams built at slight expense, or at an expense justified by some special purpose, may control the greatest possible floods, and omitting also those situations where the dry reservoir is the most practicable type, there remains an intermediate zone of great extent in which flood control and industrial or other use are competing and more or less conflicting purposes. If something can be done to make possible the common service of these conflicting purposes in future reservoir construction, the scope of flood control by means of reservoirs may be very greatly extended. Whether or not, and to what extent, this may be done will be our next inquiry.

6.—Conflict and Compromise.

In Paragraph 41 of a paper entitled "Flood Control"*, written in the summer of 1915, the writer has succinctly stated the nature of this conflict.

"If storms could be foreseen, both in date and intensity, this conflict could be compromised. But as they cannot be foreseen, and as experience shows that precipitation of one season may be several times greater or less than that of another, it becomes important, for storage purposes, to fill the reservoirs as soon as possible so as to be sure of a supply; while, for flood control, it is important to reserve ample space in them until the season of storms is safely past. The two purposes are thus essentially antagonistic."

The Ohio Valley Flood Board sets forth the same idea in its report of August 31st, 1916.† It says:

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* Read before the International Engineering Congress in San Francisco, September 24th, 1915, and published in the *Transactions* of the Congress and reprinted as Document 2, House Committee on Flood Control, 64th Cong., 1st Sess.

† House Doc. 1792, 64th Cong., 1st Sess., p. 31, par. 56.

"If, however, a system of reservoirs should be built for power purposes, they could not, even by hypothesis, be used for the prevention of floods. Power development requires that the discharge shall be fairly uniform, and shall not fall below a fixed minimum, and also that the available head of water shall not fall below a fixed minimum. * * * Regularity of flow is so important to them that whenever water is to be had they will certainly fill up the reservoir as far as they can, so as to provide for the times of small natural discharge. This is the proper way to manage power reservoirs, and it is idle to suppose that companies which have spent money for them are going to manage them otherwise."

The State Water Problems Conference, recently authorized by the Legislature of California, refers to this matter as follows*:

"In connection with this subject careful consideration must be given to the antagonistic interests of flood control, irrigation and hydro-electric power in connection with the storing of water and its use. A reservoir for highest economic efficiency in flood control should be kept empty until actual flood; it would then be filled by the first flood, and gradually emptied after that flood had subsided in order to give storage for another flood. That same reservoir, if used for power or irrigation, would, on the contrary, be filled as soon as possible—before actual flood, if conditions permitted—and kept full, lest there should not be a subsequent flood to fill it. Such a reservoir so used could not be considered, therefore, as more than a partial factor in flood control."

The Special Committee on Floods and Flood Prevention, American Society of Civil Engineers, says in this connection†:

"If a reservoir is to be utilized to diminish floods, it is essential that it be empty when a heavy rain occurs. If to increase the low-water discharge of a river, it must be full when the rainy season ceases. To insure the first condition, it is necessary to empty the reservoir after every storm, to obtain space to store the discharge of the one following. To insure the second, it is necessary to close the outlets of the reservoir at relatively low stages, and when it is once filled, not permit the surplus water to escape until the river falls to the stage when the stored water is required for navigation."

The Official Plan of the Miami Conservancy District states the case very briefly as follows‡:

"The construction of permanent reservoirs for combined flood prevention and power purposes was found not to be feasible, because

* Report of November 25th, 1916, par. 262, small pp. 85-86.

† *Proceedings*, Am. Soc. C. E., December, 1915, p. 2777.

‡ Report of Chief Engineer, February 29th, 1916, p. 84.

the same storage space cannot at the same time be used for storing water for power production, and be kept unoccupied and available for storing water in time of flood."

These expressions of opinion (and they might be extended indefinitely) indicate a well-defined conviction on the part of engineers that flood control and storage for use are, as has been repeatedly stated, "essentially antagonistic purposes." This is, indeed, perfectly true if we impose one condition, which is generally implied rather than clearly specified, that the use of the same storage space for both purposes is intended. The last citation given does clearly so state it. Under this condition no safe reliance can be placed on joint use. It might work in some cases; it will always have at least a slight favorable influence; but contingencies might at any time arise when such effect would be practicably negligible, and the reservoir be wholly ineffectual as a flood regulator. The question is not that of joint use of the same identical reservoir space, but that of having space for each purpose in addition to that required by the other. This, in fact, is the crux of the physical problem.

The possibility of securing space on the same site for both purposes is occasionally referred to in current discussions of flood control problems, but thus far without specification of details. The Official Plan of the Miami Conservancy District, for example, in the paragraph just quoted from, makes this brief reference to the subject:

"Only by creating storage space additional to that necessary for holding flood waters, can power development and flood control be provided for at the same time. In Europe such combinations have frequently worked out to advantage."

Morris Knowles, M. Am. Soc. C. E., in his Minority Report to the report of the Special Committee of this Society on Floods and Flood Prevention*, says:

"It is evident, however, that such reduced efficiency for a combination of purposes does have some value, and the maximum efficiency for each purpose can be obtained by increasing the capacity sufficiently."

Farley Gannett, Assoc. M. Am. Soc. C. E., discussing the same report, puts the whole case very clearly thus†:

* *Proceedings,* Am. Soc. C. E., December, 1915, p. 2785.

† *Proceedings,* Am. Soc. C. E., September, 1916, p. 1275.

"With few exceptions, storage in the upper part of a reservoir is the cheapest, so that, by increasing the height of dams erected for other purposes to a height greater than necessary for that primary purpose, the necessary storage for flood absorption will be obtained at more reasonable cost than if a dam or reservoir is built only for flood control."

And, later, on the same page he says:

"For these reasons the writer believes that real widespread flood control through reservoir construction and storage will be brought about by the construction of reservoirs principally for other utilization purposes, such as water supply, water power, industrial use, navigation, irrigation, and for esthetic purposes, such as park lakes, improvement, sanitation, etc."

This general idea of the combined use of a reservoir site for flood control and other purposes is characterized by the Ohio Valley Flood Board as the superposition of one reservoir upon another, and is referred to rather skeptically as follows*:

"For example (as has been sometimes proposed) a reservoir for flood prevention might be superposed on one for power development. But this would amount to having two reservoirs, since the upper part would have to be managed without reference to the lower part. Also, since the acreage overflowed increases rapidly with the height of the water, and the cost of the dam varies roughly with the square of the height, there would probably be no economy in the combination. Such superposition would therefore have to be justified by special conditions and each case be worked out on its own merit."

Likewise, the report of the Water Problems Conference of California (par. 264) refers to this subject in not very approving terms:

"Speaking generally, the same site could be used for flood control as well as for the other purposes named only by increasing the height of the dam, and having in effect two reservoirs, the top one for flood control to be filled and emptied with the coming and going of the flood, and the lower one to be filled as soon as possible and kept full as long as possible to supply water when needed. Such double construction adds materially to the cost, even if other conditions are favorable, and it must assume public control for proper use of the flood control portion of the reservoir."

The writer would bespeak for the idea thus doubtfully referred to in the last two citations a more thorough and friendly consideration than has yet been given it. May not the superposition of one reservoir

* Flood Board Report, par. 77, p. 35.

upon another contain important possibilities after all? The two reservoirs, or reservoir spaces, would be of distinct types and for distinct purposes—that below for storage, and subject to direct supervision, that above for flood control, and entirely automatic in action. The outlet of the upper reservoir would not be through conduits, as in the Miami type, but through an open spillway. Of course, in outward appearance, the two reservoirs would be one, with a single dam and a single basin above.

It will conduce to a clearer presentation of the subject if we assume what may be called an ideal example, setting forth its possible development on the foregoing lines, and then noting the qualifying conditions which must often interfere with such development, and compel its abandonment or the acceptance of something less than the highest

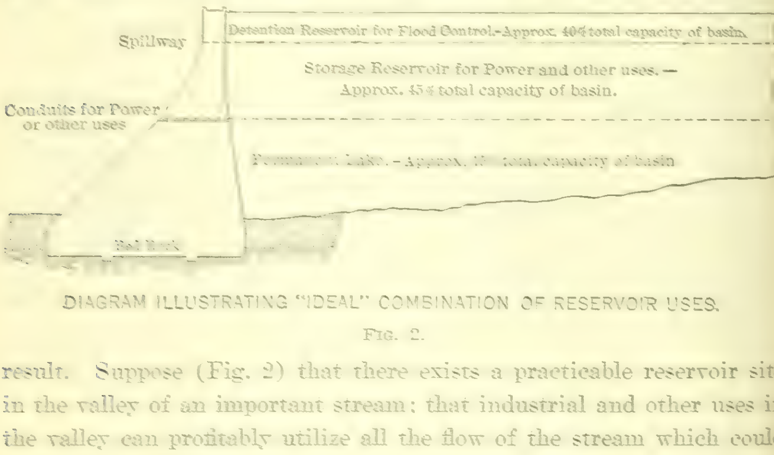


DIAGRAM ILLUSTRATING "IDEAL" COMBINATION OF RESERVOIR USES.

FIG. 2.

result. Suppose (Fig. 2) that there exists a practicable reservoir site in the valley of an important stream; that industrial and other uses in the valley can profitably utilize all the flow of the stream which could be made available; and that flood conditions in the valley are such as to justify extensive measures of relief. Let it be assumed that public authority, State or Federal, has been given the right to supervise reservoir construction, and to see that it is planned, if desirable, so as to serve all the purposes of which the development is capable.

The first consideration would be the requirements of use. It would be necessary to determine a storage capacity sufficient to equalize the flow of the stream, at least to its mean annual volume. It might indeed be very useful to do more than this, and make the excessive run-off of some years offset the deficiency of others. If it were planned to put in a power plant immediately at the base of the dam, or at least where

it could utilize the head created by that structure, it might be of advantage to make the dam permanently tight to a considerable elevation. This would insure a minimum head to be relied on at all times, and would greatly reduce the range of annual fluctuation and the resulting differences in head. With all conditions of the problem fully considered, the storage capacity would be determined, and with it the maximum level for this part of the reservoir.

The spillway would be placed at the maximum storage level just determined. The flood-control problem would then be worked out, based on the most extreme assumptions that could reasonably be made. The additional height of dam, and the spillway capacity necessary to secure the desired control, would follow, and the full dimensions of the dam would thus be determined. The portion of the reservoir below the spillway would be subject to human control; that above would not. The super-reservoir would really be of the dry type, because all that portion of the basin above the spillway level would drain out promptly after the passage of a flood, and would remain dry most of the time.

7.—Objections, Apparent and Real.

The Ohio Valley Flood Board is not strictly correct in saying* that "the upper part would have to be managed without reference to the lower part." The lower part would be managed without reference to the upper part, which would not be managed at all, but rather would manage itself—human supervision being entirely eliminated except for repairs, maintenance, and police.

As to cost, which is another matter of doubt with the Flood Board, this may be said: The capacity of a reservoir basin ordinarily increases very rapidly with its depth, or the height of the dam. In the proposed Englewood Basin of the Miami system, the following depths are required for 100 000, 200 000, 300 000, and 400 000 acre-ft. capacity: 72, 93, 107, and 119 ft., respectively. These capacities stand in almost the same ratios as the cubes of the corresponding depths, whereas, according to the Flood Board, the cost of dams increases approximately with the square of the depth. In fact, it is probably true, as Mr. Gannett has stated, that the top storage in a reservoir is generally cheapest of all.

* Report, par. [illegible]

The one serious objection of the Ohio Valley Flood Board to the detention system is contained in the following extract:*

"Detention reservoirs decrease the flood height at the expense of a prolongation of the time of high water, and therefore necessarily increase the probability of coincidence of high stages from several tributary streams."

This is theoretically true, but it is so complicated with an infinitude of other factors that it is practically impossible to determine its real value. The Flood Board seems to have in mind the Ohio River more particularly. A high flood in that stream is caused by the arrival, more or less coincidentally, of the flood waves of many tributaries. In their unregulated condition, the peaks of these flood waves are sharp, their passage of short duration, and the chance of their exact coincidence correspondingly slender. Under regulation, the peaks would be flattened out more or less, and the length of the wave increased. It is quite evident that the probability of overlapping of waves would thus be increased, but this overlapping effect would probably be more than offset by the reduction in heights. At any rate it is impossible to base a conclusion on such meager data, that flood heights would be increased.

In case only one of several tributaries were regulated, it might happen that its peak would occasionally coincide with unregulated peaks, where such coincidence would not have taken place without regulation; but the chances are just as strong the other way, and here, too, the reduction in peak due to regulation would tend to offset any unusual coincidence which might occur.

The only tangible possibility of such danger which the writer can think of would be in the case of two important tributaries in the same vicinity, one of which, as shown by long records, habitually runs out enough in advance of the other to prevent their flood peaks from coinciding. If the quicker tributary alone were regulated, it might bring their peaks more closely together, and possibly result in a higher combination; but this possibility is based on so many "ifs" and improbable conditions that it is a rather unsubstantial hypothesis after all.

The utmost that can be said of these alleged risks and dangers is that they are very remote possibilities. The probabilities are all the other way. There seems to be no sufficient ground for building up, on

* Report, par. 76, p. 35.

such a basis, an adverse sentiment toward any extension of the detention principle which is otherwise practicable. Nevertheless, the Ohio Valley Flood Board feels that the matter is of sufficient importance to call for restrictive measures. It states* flatly that it "would not recommend this type [detention] unless some means of control were provided to be used when necessary, or other Ohio tributaries were similarly regulated." Later in the same paragraph it is stated: "The Board therefore does not think that reservoirs incapable of control ought to be approved by the United States unless it can be demonstrated that the danger of increased flood heights by reason of their use is quite remote."

The writer will consider somewhat in detail the two objections specified in these citations, taking that of simultaneous regulation first. It seems quite clear to him that such regulation of a vast number of streams like the tributaries of the Ohio falls outside the domain of possibility. Any such requirement would nullify all efforts at development. The system will grow little by little—one, not many, units at a time. Better by far to take some chances and register progress than to block progress because of contingencies which are so remote as to be practically negligible.

The same caution applies as against any sweeping condemnation of the principle of automatic control. It would seem, indeed, on the face of it, that a great system of reservoirs, each subject to direct control from some central station, with a comprehensive flood-warning system giving instant data as to precipitation, run-off, gauge heights, etc., throughout the region affected, would afford more efficient control on the larger rivers than if the reservoirs were left to blind automatic operation. The matter is subject to the greatest uncertainty, however, and whatever its possible value, it would be largely offset by other considerations of direct practical importance. There would be, for instance, the immense cost of the devices necessary to control the outflow of enormous volumes of water through the dams. There would always be the risk of inefficient manipulation induced by rare use; the certainty of rapid deterioration and the likelihood of finding the mechanism not in working order when most needed; the danger of over-accumulation in the reservoir due to restriction of outflow (for that is

* Report, par. 76, p. 35.

really the form which control would take); and the probability that the interests of the valley immediately below might be subordinated to the interests, more or less hypothetical, of the trunk streams. The objection in regard to deterioration and imperfect manipulation would probably be less with spillways than with conduit outlets; but as spillways would generally be used only where the combined purposes of flood control and storage for use are contemplated, there would be ever-present a new danger in the temptation to close the outlets in the interest of private use. The writer has set forth this danger in the following terms in his Flood Control paper (par. 43) already quoted:

"As to compromise of purpose, there will always be a peril involved in the selfishness of interests dependent upon stored water. Long periods of low water dull the public sense of danger and it becomes a contest between an aggressive and insistent private appeal and a lethargic public sense of duty. The chances are strong that private appeal will prevail; and that flood reserve space will be encroached on until a catastrophic storm comes and finds the provision made for it pre-empted. This is undoubtedly a real danger."

Taking everything into consideration, the writer believes that automatic control, divorced absolutely from the possibility of human interference, possesses advantages which far outweigh any that might result from direct control.

It is well to observe at this point that the problem of flood control by reservoirs is essentially a head-waters problem. This is true also of storage for use. Reservoir development for either or both of these purposes will never produce a material diminution of flood volumes on the lower rivers. If the Miami system could be extended to every small tributary of the Ohio, floods would come to the trunk streams just the same. The local effect would be very great, the combined effect very slight; but such effect, whatever it might be, would on the whole be favorable. It would not avoid the necessity of levees and other protective works on the lower rivers; but it would reduce somewhat the stress thereon. To that extent it would operate as a factor of safety. The magnitude of that factor, however, is not sufficient to justify the building of reservoirs on the head-waters for that purpose alone. The problem will always remain essentially a local one. The writer does not believe that either adverse or favorable possible effects on the trunk streams should have a feather's weight in determining the desirability of any local development.

Perhaps most serious of the physical obstacles to the combination of purposes herein proposed is the lack of sites already referred to in Section 5 of this paper. On some streams there are no available sites; on some, such sites as are available have only capacity for storage for use; in some, a surplus capacity beyond that required for use will be available; and, on some, the full capacity for both purposes will be found. The conditions are infinitely variable, and, dependent on them, all degrees of possible control will be encountered. It should not follow that, because complete control cannot be had in a given case, such as can be had should not be accepted. If a reservoir can be made to care for 25, 50, 75, or any other percentage of the floods on a stream, such partial relief may be better than none at all. A reservoir with only capacity for its special use may at times afford effective control, and even when full, will always afford some slight control; for, no matter how large the spillway, some increase in reservoir depth is necessary to bring it into play, and to that extent it acts as a detention reservoir. Every development must stand on its own bottom, and its possibilities must be exhaustively analyzed. In that way sins of omission as well as of commission may be avoided. It is of the first importance that a site once occupied be occupied to the full extent of its possibilities. If occupied by an inadequate or inferior work, this may become a permanent bar to the highest development.

8.—The Human Factor.

Probably every engineer who has had charge of responsible work has said to himself, in the bitterness of experience, that he could cheerfully wrestle with the antagonisms of Nature if he could only escape the antagonisms of Man. So, with the problem here under consideration, great as may be the physical obstacles, those which are the direct result of human agency will be found greater still. In this final section of the paper the writer will mention, rather than attempt to remove, the chief of these obstacles.

It scarcely needs be said that to carry out any such programme as outlined herein would require that jurisdiction over our streams should be vested in some public authority. As between State and Federal control, the advantage is manifestly with the latter, because it is the one authority which embraces practically all our streams from mouth to source. It alone can exercise that uniform control which is essential

to the best results. The Ohio Valley Flood Board puts the case very strongly when it states:*

"Control over our waterways for the purposes of flood prevention and protection must, in the opinion of the Board, be delegated to one central authority before any rational plan for flood control can be devised, and compliance with such a plan, or other measures for the amelioration of flood conditions properly enforced. This central authority is logically and necessarily the Federal Government."

Next comes the matter of co-operation and development. Manifestly, the additional cost required to extend an industrial scheme in any particular case to embrace also that of flood control must be borne by some form of public agency. It would be futile to insist that private interests build larger than their immediate purposes require. If more is to be done, the public must do it, and unless it be prepared for co-operation, the desired result, no matter how practicable or important, cannot be attained. Determination of the respective shares of cost to be borne by the private interest and by the public would naturally be a bone of some contention; but, as it would be a matter of expert judgment, it should not involve serious difficulty.

The *bête noire* of the whole problem will be found in making the public fund available with promptness and certainty—unless indeed an entirely new system is devised for the public financing of such projects. When a competent body of experts, designated for the purpose, has reported that a project is feasible and deserving of prompt development, and has set forth in detail the grounds on which its findings are based, there ought to be confidence enough in its recommendations to receive the full support of Congress or Legislatures. It is known, only too well, however, that hitherto this is far from having been the case, and it is to be feared that, in this broader field of co-operation, traditional methods of delay will cause energetic co-operating interests to throw up their hands in despair. Say what we will, this is one of the drawbacks of our form of government, and the public expert can never be sure of putting through a project until he has satisfied many whose interests therein do not relate to its merits at all. It is in these and similar intangible, yet very real, obstacles, arising wholly from human wrong-headedness, selfishness, and short-sightedness, that the problem which we have been discussing will encounter its greatest difficulties.

* Report, par. 108, p. 45.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

A PHENOMENAL LAND SLIDE—SUPPLEMENT

By D. D. Clarke, M. Am. Soc. C. E.*

Synopsis.

In 1904 the writer prepared an account† of a land slide at Portland, Ore., about 30 acres in extent, which was first observed during 1894, while the construction of two of the City's distributing reservoirs was in progress, and resulted in putting these reservoirs out of service for a period of nearly 10 years.

The original paper brought the history of the slide down to the early part of 1904, and described in considerable detail the various steps taken to determine the cause of the movement, its rate of progress, and the boundaries of the moving ground, as well as the efforts made to retard the movement and counteract its effect.

The purpose of the present paper is to continue the history of the slide down to date; to describe the construction of additional drainage tunnels supplementing the original drainage system; and to note also the results of surveys made at regular dates during the intervening period, as well as to describe the character and extent of the reservoir

* This paper will not be presented for discussion at any meeting, but written communications on the subject are invited for subsequent publication in *Proceedings* and with the paper in *Transactions*.

† "A Phenomenal Land Slide," *Transactions*, Am. Soc. C. E., Vol. LIII (1904), p. 322

repair work undertaken in 1904, and successfully completed since that date, thus restoring the reservoirs and permitting of their uninterrupted use during the past 12 years.

Historical Notes.

As a preliminary to the present paper, the following brief summary of the original paper is submitted: During 1894 the Water Board of the City of Portland built two small reservoirs, each having a capacity of about 16 000 000 or 17 000 000 gal., situated in a small ravine in the City Park, about 2 miles from the business center of the city, and designed to supply the West Side, or main business district.

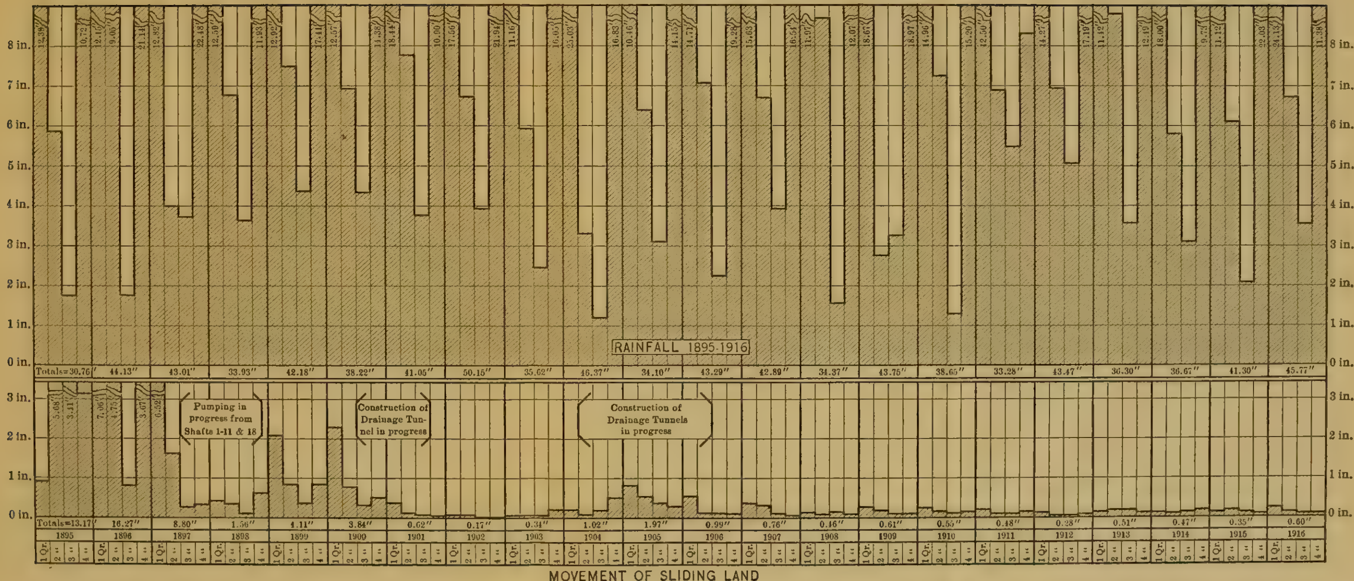
A short time before these reservoirs were completed a movement of the adjacent hillside was detected, which, at first, was thought to be entirely local and of minor importance. The work of constructing the reservoirs was thereupon pushed to completion, but the reservoir basins had scarcely been filled before the real magnitude of the movement became apparent.

The reservoir basins, therefore, were emptied at once, and instrumental surveys were promptly commenced in order to determine the extent of the slide. These surveys have since been continued at regular intervals, and by their results and a series of test borings and open shafts subsequently made and observed for a period of years (33 wash-drill borings and 22 open-shaft excavations reaching to bed-rock), the dimensions of the moving ground were at length determined to be approximately 1 700 ft. from east to west, and 1 100 ft. from north to south along the reservoir front—an area of approximately 29¼ acres—the depth ranging from 46 to 112 ft., the average being 77.8 ft. The approximate volume was 3 400 000 cu. yd., and the approximate weight 4 600 000 tons.

The borings and open shafts revealed the presence of a thin seam of blue clay along the surface of the bed-rock, with numerous water pockets in immediate connection therewith, several of the underground water pockets having considerable volume. Two of the largest of these water pockets were drained with pumps (the total pumpage aggregating several million gallons) with a marked deterrent effect on the

COMPARISON OF MONTHLY RAINFALL WITH MOVEMENT OF SLIDE

From January, 1895, to December, 1916



movement of the slide, as indicated by the periodical instrumental surveys.

Comparisons of Weather Bureau records of precipitation with the monthly movement of the slide indicated a close relationship between the two—if it did not offer absolute proof that the rate of movement of the slide depended on the volume of the rainfall during any series of months.

After a study of all the observed conditions, it at length became clear to the Water Board Engineers and the experts called into consultation, that the probable remedy was the construction of a system of drainage tunnels along the surface of the bed-rock, and that these should be located so as to tap the underground reservoirs which had been developed by the borings and open wells.

In accordance with the decision then reached, a total of 2 507 lin. ft. of such drainage tunnels, with timber supports, was constructed between June, 1900, and December, 1901, at a total cost of $14 161.14, or an average cost of $5.65 per lin. ft. for materials and labor.

The results secured by the construction of these drains were considered very satisfactory, and for a time it appeared as if the slide problem had been fully solved. That this confidence was not entirely unwarranted will appear from a study of the diagram showing the monthly rate of movement as compared with the rainfall for the years, 1895 to 1903, inclusive. This is shown as Plate XXV of the original paper, and in Plate XI the data are reproduced and extended to cover the period which has elapsed since the surveys were commenced. This diagram shows the average movement, per quarter, at approximately 50 stations along the central portion of the slide from east to west.

The volume of drainage from the tunnels was carefully observed for the 2 years following their completion, and was found to range from 10 000 to 15 000 gal. per day in summer, and from 25 000 to 75 000 gal. per day in winter; and at the end of 2 years it was decided that the drains were doing effective work and that it would be safe to proceed at once with the work of reservoir repairs.

Accordingly, in December, 1903, the writer submitted to the Water Board a report on the condition of the drainage work, stating the reasons why it appeared to be entirely safe to begin at once the work of repairing the broken reservoirs, and also submitting a plan for a permanent drain inside the tunnels already constructed.

Following the presentation of this report, the Water Board authorized an appropriation of $100 000, for reservoir reconstruction, etc., divided as follows:

For tunnel drains	$16 000
Repairs to Reservoir No. 3	36 000
Repairs to Reservoir No. 4	32 000
Roadway west of Reservoir No. 4	16 000
Total	$100 000

The fourth item covered the construction of a driveway along the west side of Reservoir No. 4, connecting with existing roadways around Reservoir No. 3.

During March, 1904, immediately following the adoption of the plan for tunnel and reservoir repairs, it was noted that there had been an accelerated movement of the slide. This was reported to the Water Board by the writer, who called attention to the unusual rainfall during the preceding 4 months, amounting to 27% more than the average for the same period during the past 21 years.

In this report the writer stated his belief that the increase in the movement of the slide, observed since the winter rains began, was due to the excessive rainfall, and that additional drainage tunnels should be constructed in order to restore the equilibrium indicated by the surveys made during the preceding 2 years. The writer also outlined a number of branch tunnels which he thought should be constructed.

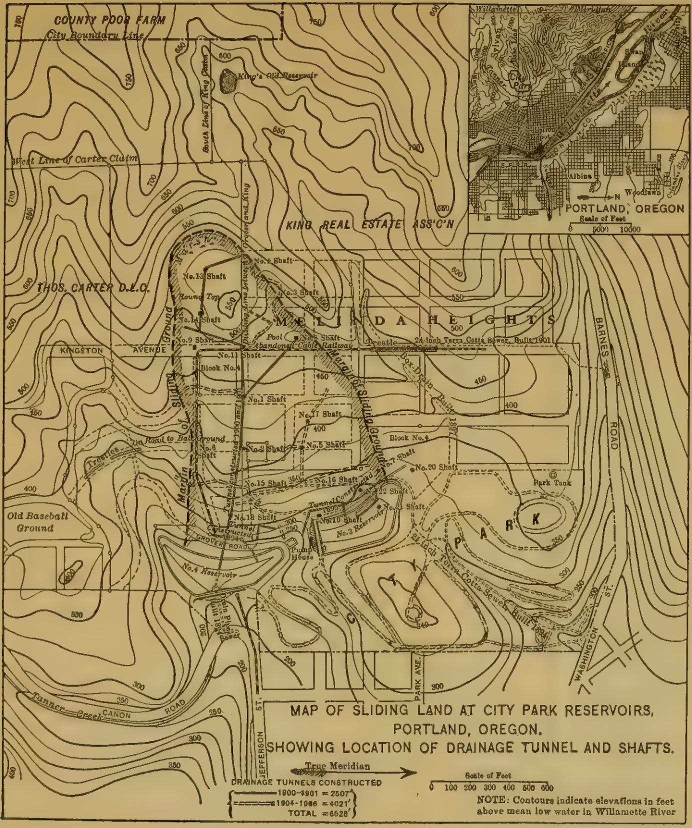
The Water Board immediately authorized the construction of these additional drains, and the work of completing them was carried forward in connection with the tunnel and reservoir repair work previously authorized.

As indicating the state of mind of the Water Board at this juncture, it is interesting to note that on March 29th, 1904, the day the foregoing tunnel extensions were authorized, the Board adopted the following resolution:

"In undertaking the repair of the reservoirs we feel the obligation to preserve the land lying west of them and to conserve the money so far expended on them, as far as we can.

"The complete system of drainage tunnels recommended by the Engineer apparently is the course to adopt.

PLATE XII.
PAPERS, AM. SOC. C. E.
SEPTEMBER, 1917.
CLARKE ON
A PHENOMENAL LAND SLIDE.



"We feel, however, that we cannot be confident that the work when completed will be absolutely permanent and without possible mishap.

"Therefore we wish at this time, for the benefit of future committees, to note that there must be constantly exercised a most careful attention and a readiness and preparation for the possible expenditure at any time of a goodly amount for repairs."

In view of the foregoing the writer is pleased to note that only nominal repairs have been required at the City Park reservoirs since the completion of the tunnel and relining work, the total amounting to only a few hundred dollars for the 12-year period which has since elapsed, during which time the reservoirs have been in continuous service.

Drainage Tunnel Extensions.

The construction of the supplemental tunnel drains already noted was commenced early in the season of 1904, and was continued until 1906, when the system of drains was completed in accordance with the revised plans.

Plate XII shows the location of these drains as related to those constructed several years earlier. These tunnels were mainly laid out from one to another of the several shafts excavated while the original exploration work was in progress.

In constructing the tunnels the excavated material was hauled from the tunnel heading to the nearest shaft in narrow-gauge cars, which were then hoisted to the surface and dumped.

The timbers used for tunnel supports, the cars and track, and the elevator cage used for this work were similar to those designed for the original tunnel project, as shown by Fig. 1.

Only a small force was employed on this work—one or two crews at different points, with sometimes two shifts per day, each crew consisting of:

One tunnel man,		\$3.00 per day of 10 hours.
Two helpers,	each	\$2.25 " " " " "
One or two top men	"	\$2.25 " " " " "
One hoisting engineer.		

The timber supports were framed by a man especially detailed for that work.

The lumber cost from \$10 to \$12 per 1 000 ft. b. m., delivered at the shaft.

A total of 4 021 lin. ft. of new tunnel was built between April, 1904, and August, 1906, at a total cost of $26 896.20, exclusive of engineering and superintendence, or an average of $6.69 per lin. ft., as compared with $5.65 per lin. ft. for work of a similar character completed in 1900-01. This increase in cost was due largely to the advance in the prices of material and labor during the intervening period. In 1901 outside laborers were paid $2.00 per day of 10 hours, and tunnel men $2.25 and $3.00 per day; and timber cost $8.50 per 1 000 ft. b. m., delivered. In 1904 and 1905 the same rate per diem was paid for labor, but the working hours were reduced from 10 to 8. This is equivalent to an advance of 25% in the cost of labor; at the same time an equal or greater advance had taken place in the price of timber and other construction materials.

The 4 021 ft. of new tunnels added to the length originally constructed gives 6 528 lin. ft. in the complete drainage system.

The material encountered in the tunnel extension work was chiefly yellow clay, intermixed with fragments of basalt. No large pockets of water were discovered, and in that respect the work did not accomplish all that was anticipated, but the aggregate volume of drainage from all the branches has been large, ranging from 18 000 to 108 000 gal. per day during some years, the quantity depending on the season of the year and the attendant rainfall. During recent years the volume of this drainage has been somewhat less than noted above.

Concrete Tunnel Culvert.

It was realized from the beginning of the tunnel work in 1900 that the timber supports would soon decay, and that ultimately a more permanent construction would have to be adopted. With this end in view, a study was made to determine the best method of lining the tunnels so as to insure the permanency of the drains.

The design finally adopted for this work was that of a monolithic concrete sewer, 28 in. in diameter, to be built entirely inside the timber frames supporting the sides and roof of the tunnel.

This plan was a modification of one adopted for a sewer built at Truro, N. S., in 1902, Messrs. Lee and Coffin being the designing engineers.* The sewer at Truro was built in an open ditch, and the arch was of brick, the chief detail of interest being the removable centering.

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* *Engineering Record,* August 30th, 1902.

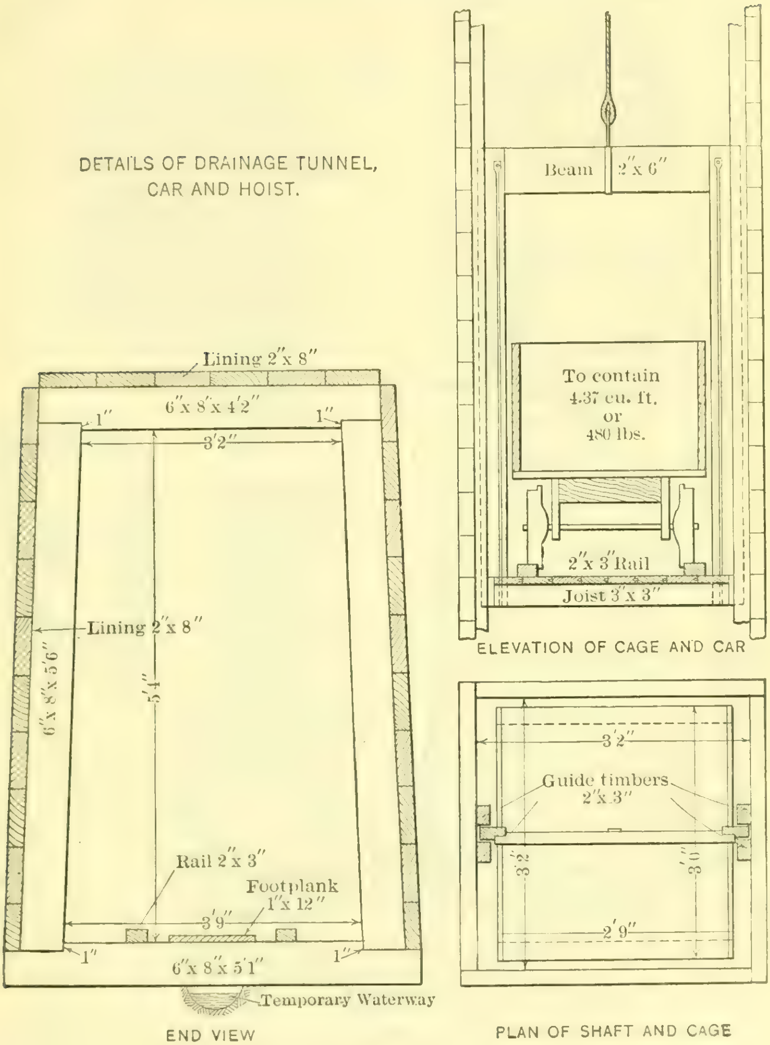
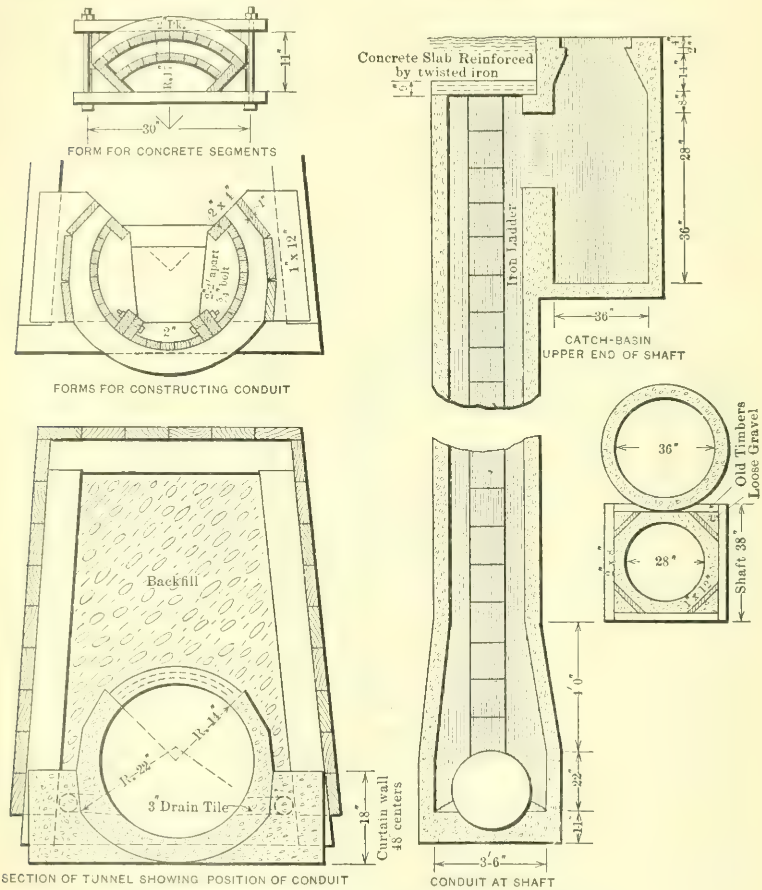

DETAILS OF DRAINAGE TUNNEL, CAR AND HOIST.
Lining 2"x 8"
6"x 8"x 4'2"
1"
1"
3'2"
Lining 2"x 8"
6"x 8"x 5'6"
5'4"
Rail 2"x 3"
Footplank 1"x 12"
3'9"
1"
1"
6"x 8"x 5'1"
Temporary Waterway
END VIEW
Beam 2"x 6"
To contain 4.37 cu. ft. or 480 lbs.
2"x 3" Rail
Joist 3"x 3"
ELEVATION OF CAGE AND CAR
3'2"
Guide timbers 2"x 3"
3'2"
3'0"
2'9"
PLAN OF SHAFT AND CAGE

Fig. 1.

For the Portland work the arch was made of reinforced concrete blocks, 12 in. in width and of sufficient length to span the opening between the side-walls. The advantages claimed for this method of construction as applied to the Portland work are two-fold:



PLAN ADOPTED FOR CONCRETE CONDUIT
IN DRAINAGE TUNNELS UNDER SLIDING LAND
TRACT WEST OF CITY PARK RESERVOIRS
(1904)

FIG. 2.

First, the base and sides of the sewer were constructed as a monolith, of the dimensions shown on Fig. 2, the distance between the side-walls permitting a man to move freely and push a car with a narrow body.

All construction materials were transported from the foot of the nearest shaft in cars running on a narrow-gauge track laid on the tunnel sills, or on the sewer invert after it had been built. The material for back-filling behind the side-walls and over the top of the sewer was hauled in the same manner.

Second, this style of construction made it possible to cast the arch blocks a sufficient time in advance to permit them to become thoroughly seasoned before being put in place, and consequently the work of back-filling was not delayed while waiting for the setting of the arch and the removal of its supports.

By placing only a few of the arch blocks in position at a time it was possible to transport the back-filling material in cars to within a few feet of the heading, thus greatly reducing the labor of shoveling and tamping required to back-fill the tunnel properly.

Fig. 2 shows the dimensions of the arch blocks and concrete sewer, and also the lining of the shafts and the connecting sump and man-holes through which access can be had to the shafts and thence to the sewer for inspection purposes, steps made of round iron rods being built into the side-walls of the shafts.

The diameter of the sewer was fixed at 28 in., that being the minimum size which would admit of comfortable inspection from end to end by a man of small or medium stature.

Of twenty-two shafts originally excavated, seven, at suitable points, were permanently lined with concrete; the others were filled with earth on the completion of the tunnel work.

In placing the arch blocks in position, the ends were cemented to the side-walls, but no attempt was made to close the crevices between the blocks at the crown of the arch. This space of, say, ¼ in. in width for every foot in length of the sewer, was left open so as to admit any seepage water which might percolate into the tunnel and thence to the top of the sewer.

At intervals of about 50 ft. a cut-off wall of concrete, 6 in. thick and 18 in. high, was built across the tunnel from side to side. These walls were deep enough and of sufficient length to cut off any flow of water along the outside of the sewer walls. The water is conducted into the sewer through a 3-in. opening left near the bottom of the invert at the up-hill side of each cut-off wall.

At each side of the sewer a line of 3-in. drain tile was laid, connecting with the opening into the sewer at intervals of 50 ft., this opening being a few inches above the upper face of the sewer invert.

The construction of the tunnel conduit was commenced in May, 1904, and the relining of the original project of 2 507 ft. was completed in May, 1905. The relining of the 4 021 lin. ft. of extension tunnels was commenced in 1908 and completed in 1910.

For the invert and side-walls of the tunnel, the concrete consisted of one part Portland cement, two parts Columbia River sand, and three parts of Willamette River gravel not exceeding 1 in. in diameter, all by volume; the arch blocks were composed of a mixture of one part of cement to three parts of concrete sand, to be mixed dry and thoroughly rammed. Two ½-in. twisted iron rods, 12 in. long, curved to the radius of the mould, were embedded in each arch block.

The construction of the tunnel conduits was carried forward by a small crew of men in connection with the work of excavating new tunnels and the repair of the reservoir linings in progress during 1904.

The concrete materials used in the work were furnished by the contractor, who delivered materials of the same class for the reservoirs; the mixing and laying of the concrete were done by day's labor under the direction of the Department foreman. The tunnel foremen were paid $3.00 per day, and other inside labor $2.25 per day.

Detailed costs, kept during the period from June 1st, 1904, to June 1st, 1905, showed that 2 746 lin. ft. were completed at an average cost of $3.20 per lin. ft. for the materials and labor for constructing the conduit and back-filling the tunnel.

Repair of Reservoirs Nos. 3 and 4, in 1904.

One effect of the slide had been to shatter badly the concrete lining on the western slopes of both reservoirs, and the problem was to replace this lining, and also to reinforce the old lining on the bottom and on the eastern slopes of the reservoirs, so as to insure that the basins should be thoroughly water-tight.

As originally built, the lining of these reservoirs consisted of cement concrete placed on the bottom and sides of the basins, the sides having been graded to a uniform slope of about 1 to 1½. In places where the material forming the slopes was of a rocky character a

layer of clay and gravel puddle, 8 in. thick, had been placed under the concrete. For the purpose of supporting and anchoring the lining to the slopes, a network of ¼-in. twisted iron rods was embedded in the concrete about midway between the upper and lower faces. These rods and extended up and down the slopes. The trenches in which they were anchored to the tops of concrete posts, about 8 in. in diameter and 3 ft. long, moulded in place in the slopes.

A system of 4-in. tile drains under the western slopes also formed a part of the original plan. These drains were about 30 ft. apart, and extended up and down the slopes. The trenches in which they were placed were about 12 in. deep, and were filled with coarse sand after the tiling was laid.

During the few months the reservoirs were in use in 1895 it was demonstrated beyond question that the original linings were not as effective as they were designed to be. The case was a complicated one, for, in addition to the original thickness of the concrete and the underlying layer of puddle or compacted earth, the concrete was laid in sections with an expansion joint ½ in. wide running up and down the slopes at intervals of from 12 to 20 ft. For the lower half of these joints the concrete of adjoining sections was laid in close contact, but without attempting to form a thorough bond; the upper half of the joint was filled with asphalt or asphaltic mastic, the entire upper surface of the lining then being covered with a mop-coating of hot asphalt.

Considering the length of the slopes—from 50 to 80 ft.—and the extreme depth of the basins—41 ft. behind one dam and 49 ft. at the other—the problem of making repairs that would be effective was one that caused the writer no little anxious thought. Nowhere in the literature of engineering within his reach at the time could he find any account of work of a similar character, undertaken under conditions nearly as severe as existed here. These conditions may be enumerated as follows:

1.—The extreme pressure to which the linings would be subjected (maximum pressure, 22 lb. per sq. in.);

2.—The long slopes without a break; and

3.—The necessity for providing for a water-proofing course which of itself should possess sufficient strength and elasticity to span any small opening in the foundation layer of concrete,

which might be caused by a renewed or spasmodic movement of the slide.

The apparent necessity for relining the entire surface of each basin also added to the expense of the work.

Without describing in detail all the steps of the evolutionary process by which a conclusion was reached regarding the method to be adopted for making these repairs, it may be stated briefly that it was decided:

First.—That the original concrete lining on the west slopes of both reservoirs should be removed where broken or unsound and the banks dressed down; that the old tile drains should be cleaned out and additional under-drains constructed, the trenches to be filled with gravel and broken stone, instead of sand. (At one point on the west slope of Reservoir No. 4 the movement had reached a maximum of 3 ft., making necessary a re-grading of the slope for a considerable distance.)

Second.—That a new base course of concrete, having a thickness of 7 in. at the base of the slope and 5 in. at the top, should be placed on the slopes thus prepared.

Third.—That on top of the new layer of concrete, and on the top of the old lining covering the bottom and east slopes, there should be placed a layer of water-proofing material of sufficient strength to insure that it would bridge over any small cracks which might subsequently develop in the foundation course.

Fourth.—That on top of this water-proofing course there should be placed a second layer of concrete, 6 in. thick at the base of the slopes and 4 in. at the top; and that the supplemental layer on the bottom of the reservoir basin should have a uniform thickness of 5 in. At the foot of the slopes the concrete layers were to be made thicker, so as to round off the angle between the sides and the bottom. This is shown by the cross-section of No. 3 reservoir basin, Fig. 3. The rectangular footing, about 2 by 3 ft., shown on the plans, was only extended along the west slope of Reservoir No. 3.

In carrying forward the repairs of the reservoir basins, the following procedure was adopted:

The removal of the broken concrete lining on the west slopes of both reservoirs was done by Water Department forces, the material being piled at convenient points outside of the basin, and removed later and utilized for paving roads in the adjacent Park.

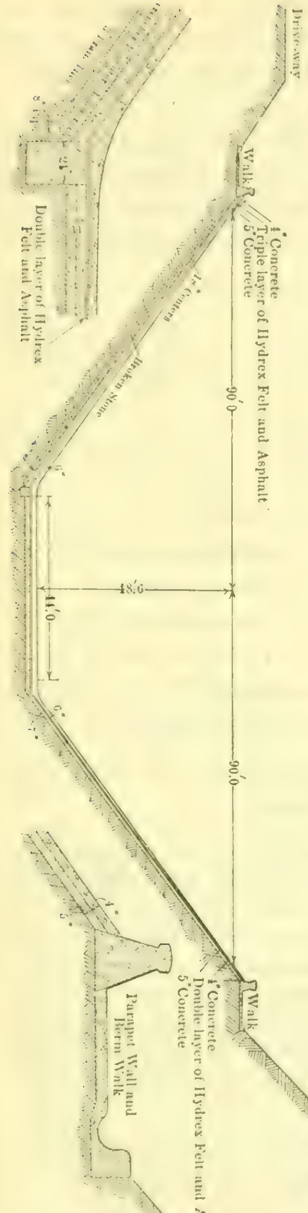
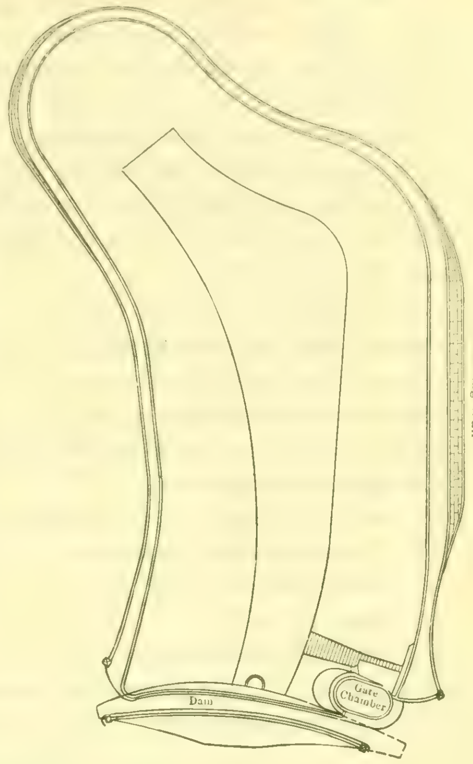
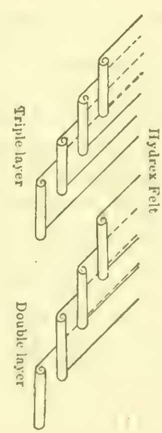


FIG. 3.





ADOPTED PLAN
OF WATER DEPARTMENT, PORTLAND, ORE.,
FOR RELINING AND WATER-PROOFING
RESERVOIR NO. 3 (AND 4), CITY PARK,
IN 1904.

Contracts were entered into by the Department for purchasing and hauling lumber, cement, sand, gravel, and crushed rock, also for asphalt and felt. Other supplies in smaller quantities were purchased on requisition.

Contracts were also made covering the labor of mixing and placing the concrete, and for laying the felt and asphalt water-proofing, the materials being furnished by the City at the site of the work.

The prices paid for materials and labor required for the reservoir repairs were as follows:

Cement, $2.40 to $2.65 per bbl.

Broken stone, $2.25 to $2.82 per cu. yd.

Gravel, $1.45 to $1.85 per cu. yd.

Sand, $1.45 to $1.85 per cu. yd.

Felt, f. o. b. factory, $1.25 per 100 sq. ft.

Asphalt, $16.00 per ton.

Labor, laying concrete (all materials furnished):

Foundation and top course,	$2.75 per cu. yd.
Cement finish,	$0.15 per sq. yd.
Parapet wall,	$3.00 per cu. yd.
Laying felt: first layer,	$0.35 per 100 sq. ft.
" " second and third layers,	$0.25 " " " "
Hauling cement,	$0.095 per bbl.

The following extract from the specifications for the work shows the proportions adopted for the concrete mixture.

"In proportioning materials for mortar, grout, and concrete, one volume of cement shall be taken to mean 380 lb. net. One volume of sand, gravel or stone shall be taken to mean 3.7 cu. ft. packed or shaken down. Sand, gravel, and broken stone shall be measured in barrels or boxes. Measurement in wheel-barrows will not be permitted.

"The concrete for the foundation course on the west slope shall be proportioned as follows: 1 part cement, 3 parts sand, and 6 parts broken stone or gravel. For the surface layer of concrete the proportions shall be, 1 part cement, 2½ parts sand, and 4 parts gravel or stone, as the Engineer may direct."

It may be stated that broken stone was used in the concrete forming the base course on the west slope, resting on the earth, but for the upper layer, which rested on the felt water-proofing, gravel was substituted for broken stone in order to avoid the danger of puncturing

the felt water-proofing course while the concrete was being rammed into place.

Some difficulty was experienced at first in holding the upper layer of concrete in place on top of the water-proofing course. Owing to the steep slope, it was impossible to do much tamping, and if a slight excess of water was used there was danger of the whole mass moving down the slope. In fact, during the first day or two, such slides did occur, making it necessary to relay portions of several sections. In several instances a slight movement of the concrete occurred after the entire section had been placed, caused by the weakening of the footing. This difficulty was finally obviated by placing the concrete on the bottom of the reservoir and up the sides of the basin to a height of 5 or 6 ft. and allowing it to harden before completing the upper portion of the section.

Fig. 3 shows a cross-section of Reservoir No. 3 near the dam. The two courses of concrete, with the water-proofing course between them, and the tile and rock drains, are also shown.

The method adopted for the repair of Reservoir No. 4, being identical with that used at Reservoir No. 3, work on both basins was carried forward under the same contract. After the completion of the concrete work, the surface of the linings of both basins was painted with a thin coating of asphalt in order to give the work a uniform color.

Water-Proofing.

The best method of water-proofing the reservoir lining was made a study for a considerable period. It was assumed that possibly a slight movement of the hillside might continue for some time longer, and it was considered desirable, therefore, to use some sort of membrane water-proofing having sufficient strength and elasticity to permit it to bridge cracks in the concrete base in case the latter should again be fractured under pressure from the adjacent hillside.

In the method of water-proofing finally adopted two layers of "hydrex felt" and three coats of asphalt were used for the bottom and eastern slopes of both reservoirs, with three layers of felt and four coats of asphalt for the western slopes, where the original lining had been so badly shattered by the pressure of the slide.

This water-proofing membrane method practically conformed to that adopted by the Pennsylvania Railroad engineers for water-proofing

the Hudson River Tunnel, then under construction, except as to the number of layers of felt, and the substitution of asphalt as a binder in place of coal-tar pitch specified for the Pennsylvania Railroad work. The change in the binder coat was made on the suggestion of the late Alfred Noble, Past-President, Am. Soc. C. E., Consulting Engineer for the Pennyslvania Railroad work, because it was thought asphalt would make the membrane more elastic and hence more suitable.

In order to demonstrate the effectiveness of a membrane lining similar to that just described, under conditions comparable with those under which it was to be used, it was decided to experiment with a typical section before the material was finally adopted. Accordingly, a small section of such a membrane, about 1 sq. yd., was built up for testing purposes. This consisted of two layers of felt, each coated with asphalt on both sides, which was subjected to a water pressure of 50 ft.—the maximum working pressure at Reservoir No. 3. The water chamber used for testing purposes, to which this pressure was applied, was of cast iron, except that one side consisted of fir planks 3 in. thick and 12 in. wide, with a space of ½ in. between adjacent planks. The section of membrane to be tested was placed on the inner face of these planks, and was supported by them, the membrane spanning the ½-in. opening between the planks. The joints around the margin of the chamber were sealed with a soft rubber packing.

During the test, which was continued for nearly 6 months, it was found that under pressure the membrane was forced into the cracks between the planking, and for a period of 146 days did not weaken sufficiently to allow the water to escape from the 6-in. stand-pipe supplying the pressure. It was concluded, therefore, that the material was of sufficient strength and elasticity to answer the purpose in view.

It may be said, further, regarding this test, that after the pressure had been maintained for 146 days it was released for 119 days, at the end of which time it was again applied and the test continued. At the end of a further period of 44 days, a slight crack, a few inches in length, developed in the felt where it had been compressed into the space between the planks, thus allowing the water to escape from the stand-pipe and release the pressure. The actual test, therefore, covered a period of 190 days.

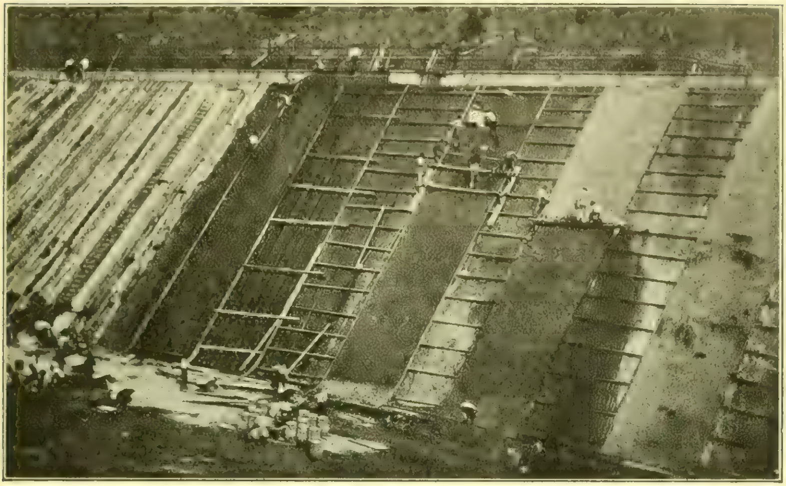


FIG. 4.—RELINING EAST SLOPE, RESERVOIR NO. 3, SEPTEMBER 13TH, 1904.

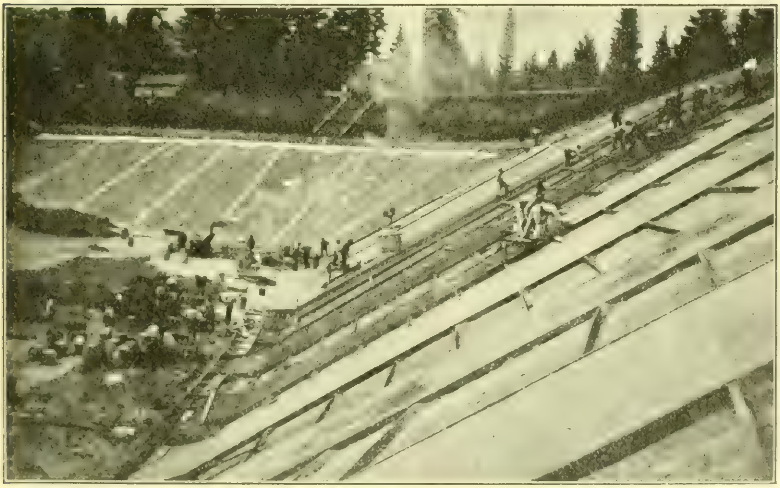


FIG. 5.—RELINING EAST SLOPE, RESERVOIR NO. 3, SEPTEMBER 13TH, 1904.

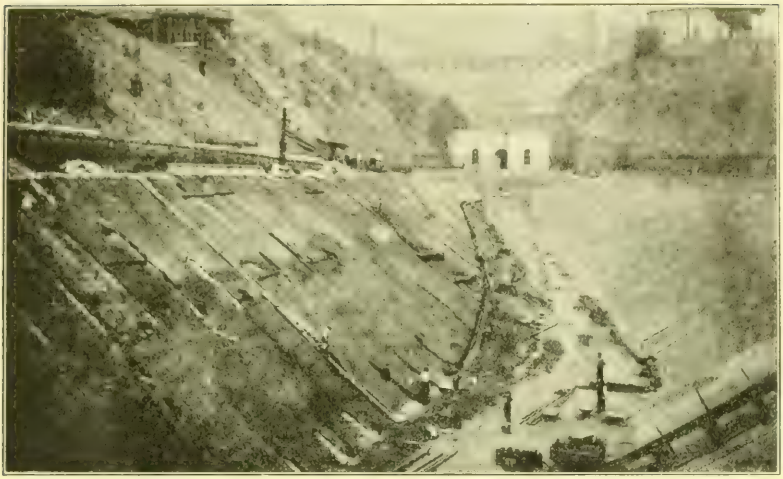


FIG. 6.—WEST SLOPE, RESERVOIR NO. 4, SHOWING UNDER-DRAINS, SEPTEMBER 21ST, 1904.

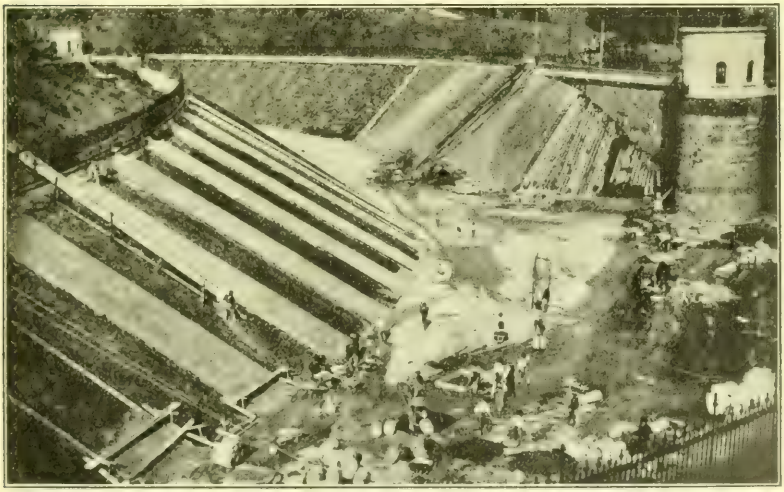


FIG. 7.—RELINING WEST SLOPE, RESERVOIR NO. 4, OCTOBER 17TH, 1904.

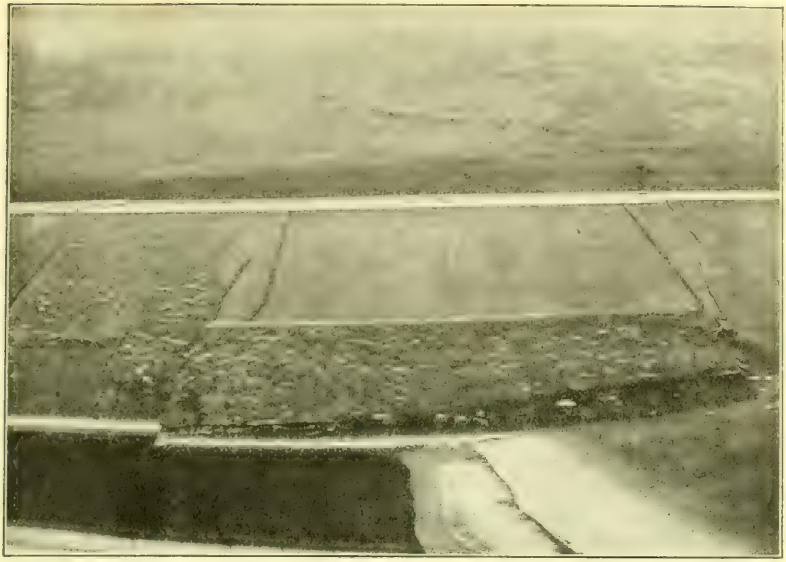


FIG. 8.—RESERVOIR NO. 3: HORIZONTAL CRACKS IN BUTTRESS, AND BREAKS IN WEST SLOPE AND PARAPET, SEPTEMBER 28TH, 1897.

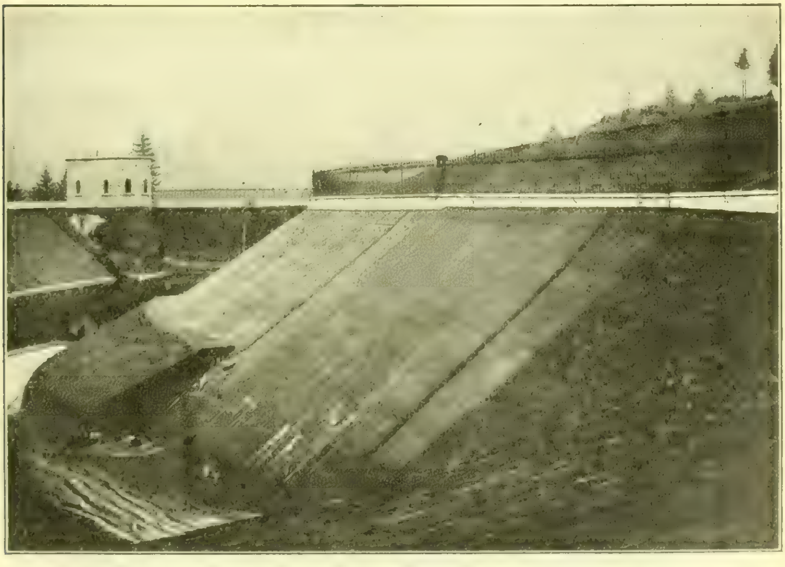


FIG. 9.—RESERVOIR NO. 2: CRACKS IN BUTTRESS AND IN PARAPET, SEPTEMBER 28TH, 1897.

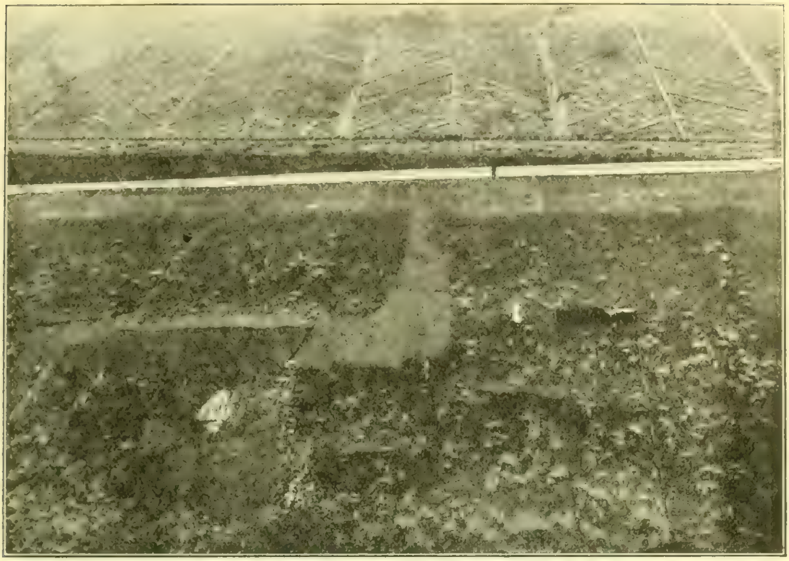


Fig. 10.—Reservoir No 4: Cracks in Parapet Wall and West Slope, September 28th, 1897.

Fig. 11.—Reservoir No. 4: Crack in Outer Edge of Inclined Roadway, and in Face of Slope Above Sub-Retaining Wall, September 26th, 1897.

Fig. 12.—Reservoirs Nos. 3 and 4, Portland, Ore., Before Repairs Were Completed in 1904.

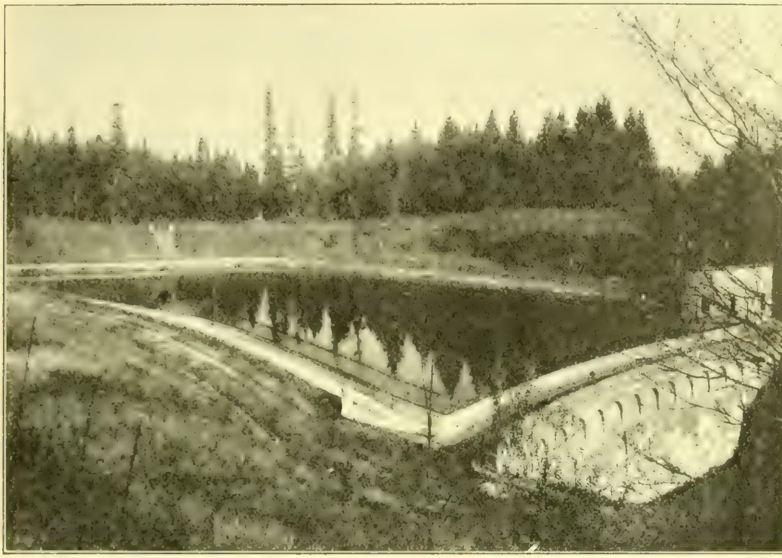


Fig. 13.—Reservoir No. 3 Since Repairs Were Completed in 1904.

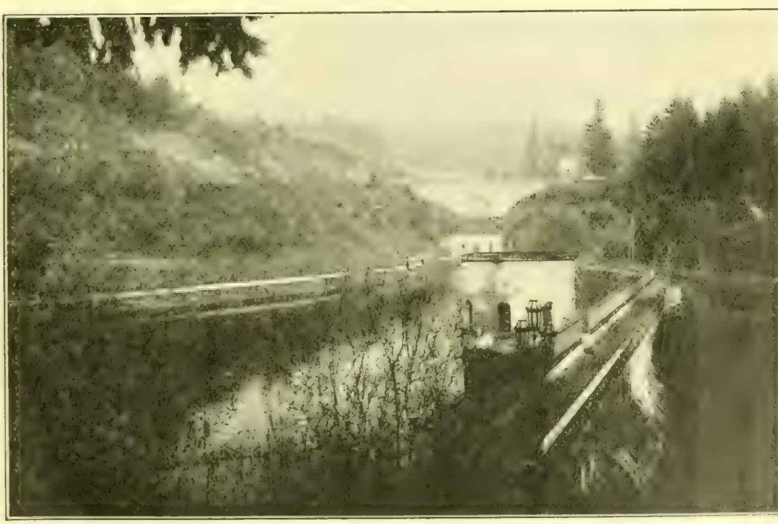


Fig. 14.—Reservoir No. 4 Since Repairs Were Completed in 1904.

A small section of this membrane, showing the crack which developed under the foregoing test, was cut out at the conclusion of the work, and has since been preserved. A recent examination shows that at present this sample of the membrane lining has a considerable degree of flexibility, notwithstanding the fact that it has been stored in a dry and warm room for 12 years.

In laying the sheets of felt, they were lapped from 2 to 4 in. at their edges, and from 6 to 12 in. at their ends, there being three layers of felt for the west slopes and two layers for the remainder of both basins.

Regarding the effectiveness of the water-proofing work and the condition of the reservoirs at the present time, the following may be stated: When the basin of Reservoir No. 3 was first filled, in March, 1905, after the relining was completed, a slight seepage through the under-drain was observed when the water behind the dam was only about 10 ft. deep, the slope of the reservoir bottom being such that the flow line did not then extend more than about half the length of the basin. The system of drains under the lining centers in a single 6-in. pipe which passes through the base of the dam, the flow in this pipe being controlled by a gate in a chamber in front of the dam. The measured flow from this drain, when first observed, was only at the rate of about 8 000 gal. per day. As the water in the basin arose, the flow from the drain increased, due to the additional head. This has been the case every time the reservoir has been emptied and refilled; for several years the maximum flow when the reservoir was full has been about 36 000 gal. per day; and, for the last year or more, it has ranged from 55 000 to 60 000 gal. per day.

No serious attempt has been made to repair this leak, but it is thought to be near the dam, if not at the joint between the lining and the vertical face of the dam.

The loss of water is not a serious matter at present, and hence the question of further repairs to the lining has not been considered important. It can be said that this is the only defect in the work undertaken at Reservoir No. 3.

At Reservoir No. 4 no such leakage has occurred. At this basin, however, a slight cracking of the west parapet wall has been detected, and one or two of the fence posts are now inclined outward. A 6-in.

space was left between the top of the slope lining and the parapet wall, and this has not yet been closed by the earth pressure.

Seepage through the concrete dams has been in evidence for some years and, in consequence, the outer faces of both dams of Reservoirs Nos. 3 and 4 have become much discolored from the laitance deposits (as appears from an examination of Figs. 13 and 14), no attempt having been made to water-proof the inner or water face of the dam at either reservoir in connection with the relining of the slopes.

Figs. 4 to 7 show the workmen at Reservoirs Nos. 3 and 4 engaged in laying and coating the hydrex felt and mixing and placing the concrete covering at different stages of the work.

Figs. 8 to 14 show the reservoir linings before repairs were commenced and the basins and dams as they now appear.

During 1894, the late J. D. Schuyler, M. Am. Soc. C. E., was the Consulting Engineer in charge of the design and construction of the four reservoirs then being built by the City, including the City Park Reservoirs, Nos. 3 and 4, described herein.

The reclamation work described in the original paper was carried on under the direction of the late Isaac W. Smith, M. Am. Soc. C. E., who was Chief Engineer until his death on January 1st, 1897, the writer being Principal Assistant Engineer during this period. Since that date the writer has been Chief Engineer in charge of the work herein described.

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PAPERS AND DISCUSSIONS

HYDRAULIC PHENOMENA AND THE EFFECT OF SPREADING OF FLOOD WATER IN THE SAN BERNARDINO BASIN, SOUTHERN CALIFORNIA

By A. L. Sonderegger, M. Am. Soc. C. E.

To be Presented October 17th, 1917.

Synopsis.

The San Bernardino Basin is a closed structural basin filled with detrital matter, in which Artesian conditions are created by an impervious barrier thrown across its outlet. The barrier is locally known as Bunker Hill Dike. Geologically, these conditions are the result of block faulting. (See Plate XIII.)

There are marked wet and dry seasons, and a wide range in fluctuations of seasonal as well as periodical precipitation. This is demonstrated by the residual mass curve of rainfall, constructed for a 45-year period, which curve, in its ascending and descending branches, indicates that there have been four distinct periods, of from 10 to 13 years each, of excessive and deficient rains. (Figs. 1 to 4.)

Broadly speaking, variations in precipitation call for corresponding variations in the run-off of streams tributary to the basin. (Figs. 4 and 7.)

Under conditions of natural draft, ground-water levels fluctuate in sympathy with precipitation and run-off. There is a well-defined rela-

Note.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and, when finally closed, the papers, with discussion in full, will be published in *Transactions*.

tion between the elevation of the Artesian rim of an Artesian basin of this character, and the quantity of rising water in streams and Artesian wells, and pressure in such wells. (Figs. 5 and 6.) The theoretical head at any point in the Artesian basin is equivalent to the difference in topographical elevation between that point and the Artesian rim.

The Artesian phenomena of pressure and flow are in direct sympathy with the rainfall and run-off phenomena, as expressed in the residual mass curves. The residual mass curves of rainfall and run-off afford a safe criterion as to the position which the ground-water plane naturally should occupy, and whether artificial abstractions have materially overdrawn a basin. (Figs. 7 to 14.)

The maximum natural seepage in the river bed of the Santa Ana River occurs for a mile or two above the Artesian rim. (Table 2.) The effect of artificial seepage applied near the rim is but temporary, and the benefits increase the farther up on the débris cone the spreading is practised. The center of the spreading works on the Santa Ana cone, of late years, has been about 4 miles above the Artesian rim, and this spreading has had the effect of diminishing the seasonal drop of the water-plane above the rim by 3 or 4 ft. per year. Contrary to common opinion, it is beneficial to practise spreading, even during years of average or deficient rains, when practically no flow would escape beyond the limits of the basin, provided the application is made in the upper parts of the débris cone. (Figs. 15 to 23.)

The cost of spreading flood water varies from 5 to 20 cents per acre-ft., a maximum of 27 000 acre-ft. having been spread in one year. The average annual net conservation is estimated as in excess of 15 000 acre-ft. per year, and valued at not less than $100 000; this has been accomplished at an average annual expenditure of less than $2 000.

The San Bernardino Basin, about 70 miles east of Los Angeles, is a closed structural basin bordered on the north, east, and south by granitic and schistose mountains. To the west the valley is open, but the underground basin is closed by a subterranean barrier, known locally as the Bunker Hill Dike. Plate XIII is a general map of the basin. The dike consists of impervious clays which effectively close the basin and force practically all the underflow to the surface. The


GENERAL MAP OF
SAN BERNARDINO BASIN
CALIFORNIA
SAN BERNARDINO MOUNTAINS
Little Mountain
San Bernardino
Perris Hill
Meacham Well
Heap Well
Mc Crary Wells
Bledsoe Well
Butmaster
Slack
Dobbs
Taylor
Bradley
Kellthum
Frates
Shick
Huston
Schoolhouse
Williams Well
Spreading Grounds
Richey Well
Beech Well
Gage Canal Co's Wells
Bunker Hill
Colton
Slover Mt.
City of Riverside
Redlands
Badlands Mountains
Lytle Creek Faultline or Bunker Hill Dyke
Santa Ana River
Mill Creek
City Creek
ARTESIAN RIM
HIGHLAND AVE.
BASE LINE ROAD
SAN BERNARDINO AVE.
COLTON AVE.

basin is filled with alluvial deposits of gravel, sand, and clay to unknown depths, the deepest borings going to 1 150 ft. without encountering bed-rock. The deposits are porous on the débris-cones and receptive to the absorption of flood waters, or irrigation water. They are graded from coarse gravels and boulders, at the mouths of the canyons, to fine silts in the flats.

The Santa Ana River has cut a gap in the dike to a width of from 1 to 1½ miles, in which the impervious strata reach probably to within 100 ft. of the surface; and, on each side of it, the dike manifests itself as a ridge projecting above the valley floor. The effect of the dike has been to produce an Artesian basin, from which large quantities of water rise to the surface and escape, either in rising streams, or by evaporation from swamps. The maximum area of the Artesian basin in the early Nineties was 21 sq. miles. The water-shed tributary to the Artesian basin is 720 sq. miles, of which 560 sq. miles are mountainous water-sheds and 160 sq. miles are foot-hills and valley lands. The average annual water crop amounts to about 280 000 acre-ft., and the annual use is about 200 000 acre-ft.

Of the areas irrigated, there are within the basin 26 100 acres of citrus lands in the Redlands and Highland districts, and 5 000 acres of valley land planted to alfalfa and garden truck; outside, there are the Riverside Colony, with 27 700 acres, mostly in citrus fruits, and the Rialto and Fontana districts, with about 16 000 acres, a large percentage of which is also in citrus groves.

The total value of the land depending on the San Bernardino Basin for its water supply is in excess of $100 000 000.

All summer flow of the surface waters has been appropriated for many years, and during the last 20 years increasing quantities of water have been abstracted by Artesian and pumped wells. More than 50% of the water is diverted beyond the limits of the San Bernardino Basin. Notable among the concerns which export water are the Riverside Water Company, the Gage Canal Company, the Riverside-Highland Water Company, the City of Riverside, the Lytle Creek Water and Improvement Company, the Fontana Company, and others.

The underground supply of the basin is mainly derived from the seepage that occurs in the stream beds in the upper part of the débris cone between the mouth of the canyon and the rim of the Artesian basin. Except during years of extreme drought, large quantities of

flood water escape during the flood season, beyond the Artesian rim, and beyond the Bunker Hill Dike, and are forever lost to the San Bernardino Basin.

Recently, owing to an action in the Courts filed by the City of San Bernardino against the City of Riverside and the Riverside Water Company, the hydrography of the basin and the effect of the spreading of flood waters have been made the subject of a detailed study, the writer having been one of the engineers engaged by the defendants. In order to eliminate unnecessary details, the scope of this paper will be limited to the eastern portion of the basin, which may be termed the Santa Ana cone, embracing the drainage areas and débris cones of the Santa Ana River, Mill Creek, Plunge Creek, and City Creek.

The Residual Mass Curves of Rainfall and Run-off.

There are no local rainstorms in Southern California, and the Weather Bureau records at San Bernardino may be considered typical of the distribution of rainfall in the San Bernardino Basin and watershed, relative to both time and volume. With higher altitudes there is a proportionate increase of precipitation, up to the elevation of 6 000 ft.

TABLE 1.—Monthly Precipitation at San Bernardino, California.

Month.	Depth, in inches.	Percentage of seasonal precipitation.
January	3.708	22.9
February	3.005	18.6
March	2.764	17.1
April	1.222	7.5
May	0.570	3.5
June	0.081	0.5
July	0.035	0.2
August	0.172	1.0
September	0.163	1.0
October	0.600	3.7
November	1.342	8.3
December	2.532	15.7
46-year average	16.194	100.0

The distribution of the precipitation for the year, at San Bernardino, is shown in Table 1. There is a marked difference between wet and dry seasons. The rainy months are from December to March, inclusive, during which 74% of the seasonal rainfall occurs. A tabu-

lation of seasonal rainfall at San Bernardino, from 1871 to 1916, inclusive, is shown on Figs. 1 to 4. It is difficult to grasp the general tendency of annual variation of precipitation from tables, nor are the periodical fluctuations easily discerned. In order to overcome this, residual mass curves of precipitation have been prepared for three periods: from 1870 to 1900, 1870 to 1910, and from 1870 to 1915. (Figs 1, 2, and 3.)

These curves are a diagrammatic presentation of aggregate excesses and deficiencies of precipitation; an ordinate at any point on one of them is the difference between the aggregate rainfall from the beginning of the period of record to the date considered, and the aggregate rainfall which would have fallen during the same period if the annual rate had been the observed mean.

Such a curve is partly above and partly below the zero line. It always begins near the zero line, and necessarily closes on the same, regardless of the length of the period considered. For this reason the position of the zero line is of no consequence as regards the interpretation of the curve. The curve is in descent for years of deficient precipitation, and in the ascent for years of excessive rains. If the deficiencies extend over a period of years, this is expressed in a general downward tendency, as from 1874 to 1883, and from 1893 to 1904, and *vice versa*. Thus, it will be seen that there are four distinct periods from 1874 to 1916, as indicated at the head of Figs. 1 to 4.

These mass curves, therefore, indicate the cumulative effect of precipitation, expressed in ascending and descending branches of the curves. An analysis made at the close of the season of 1899-1900, as shown on Fig. 1, would have indicated that there had been a deficiency in precipitation for the 7 preceding years, but it would not have indicated that in 1900 anything like average conditions had been reached. Compare the different positions of the point marked 1900 in Figs. 1, 2, and 3.

Fig. 4 presents the residual mass curve of run-off for the Santa Ana River for 46 years. It shows the same characteristics as the rainfall mass curve, and calls for a like interpretation. The observations from 1897 to 1916 are from measurements made by the Geological Survey; the volumes given for the years 1871 to 1897 are deduced from run-off curves. Generally speaking, the fluctuations of the run-off mass curve correspond to those of the rainfall. Deviations are due to

the fact that, for equal rains, the run-off is greater if the preceding year was one of excessive precipitation than if it was a dry year. The relation which variations in rainfall bear to those of run-off is best studied by platting the residual mass curves for both, as expressed in percentage of the mean. This is presented in Fig. 7. In this diagram the run-off mass curve represents the entire natural water crop tributary to the San Bernardino Basin, the rainfall being that for the City of San Bernardino only. It will be seen that the functions in run-off are from 50 to 100% greater than those of corresponding precipitation, both for maxima and minima.

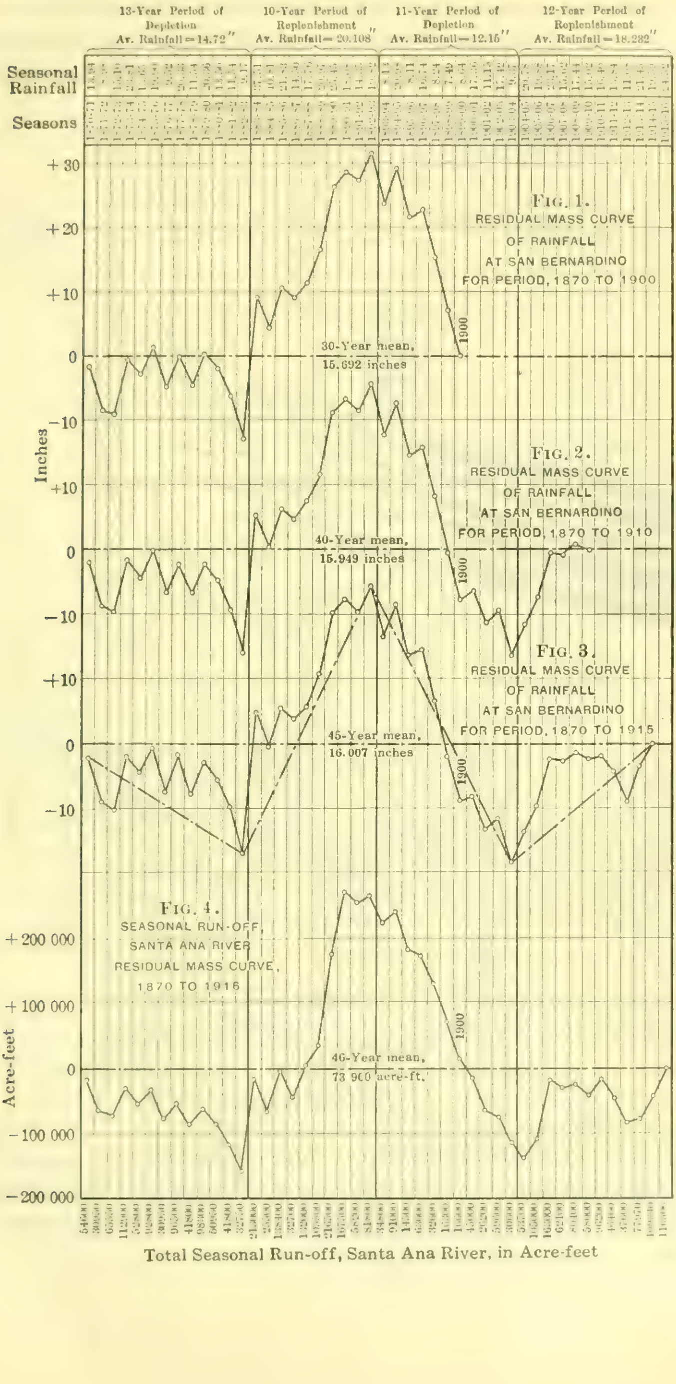
Ground-water levels generally respond to variations in rainfall and run-off, and, broadly speaking, should hold their own for average rains. A series of years of deficient rains would be accompanied by deficient replenishment and the corresponding lowering of the ground-water level, though nothing short of a continued period of excessive rains would bring about a recuperation of a depleted basin.

The residual mass curve of rainfall or of run-off, therefore, may be considered as a reliable indicator of the status of the ground-water level, provided conditions of natural draft obtain. Furthermore, if the water level of an underground basin, which is artificially drawn upon, corresponds to the position indicated by the mass curve of precipitation or run-off, then it may be deduced that the basin fluctuates in the same manner as under natural conditions, and, therefore, is not being overdrawn.

Considering the wide range in fluctuations of the seasonal precipitation in the San Bernardino Basin, ground-water levels should be subject to extensive variations from natural causes. These statements will be supported by facts given in the pages following.

The Movement of Underground Water.

If the San Bernardino underground basin had an unobstructed outlet to the west, it would produce the common phenomenon of underflow, with velocities of percolation of probably 2 miles per annum, or thereabouts. Any material contribution to such underflow at any point by means of spreading, would produce what might be called an advancing wave of underflow, and this, in due time, would be noticeable at points below by a rise in the water surface, a fact which could easily be established by observations on a series of wells below the spreading


13-Year Period of Depletion Av. Rainfall = 14.72"
10-Year Period of Replenishment Av. Rainfall = 20.108"
11-Year Period of Depletion Av. Rainfall = 12.15"
12-Year Period of Replenishment Av. Rainfall = 18.282"
Seasonal Rainfall
Seasons
Inches
FIG. 1. RESIDUAL MASS CURVE OF RAINFALL AT SAN BERNARDINO FOR PERIOD, 1870 TO 1900
30-Year mean, 15.692 inches
FIG. 2. RESIDUAL MASS CURVE OF RAINFALL AT SAN BERNARDINO FOR PERIOD, 1870 TO 1910
40-Year mean, 15.949 inches
FIG. 3. RESIDUAL MASS CURVE OF RAINFALL AT SAN BERNARDINO FOR PERIOD, 1870 TO 1915
45-Year mean, 16.007 inches
1900
FIG. 4. SEASONAL RUN-OFF, SANTA ANA RIVER RESIDUAL MASS CURVE, 1870 TO 1916
46-Year mean, 73 900 acre-ft.
Acre-feet
+ 200 000
+ 100 000
0
− 100 000
− 200 000
Total Seasonal Run-off, Santa Ana River, in Acre-feet

works. One would also be justified in crediting the underground basin with the total quantity of water spread. However, such conditions do not exist in the eastern part of the San Bernardino Basin.

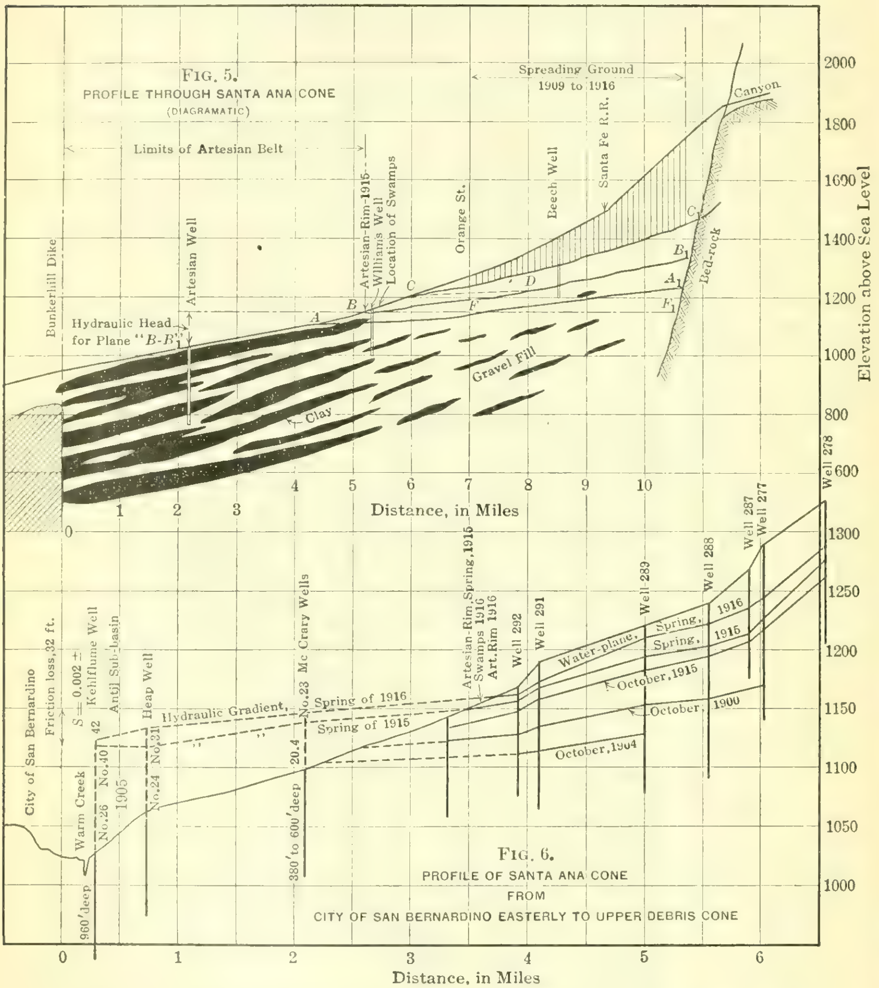


FIG. 5.
PROFILE THROUGH SANTA ANA CONE
(DIAGRAMATIC)

FIG. 6.
PROFILE OF SANTA ANA CONE
FROM
CITY OF SAN BERNARDINO EASTERLY TO UPPER DEBRIS CONE

Fig. 5 is a profile of the Santa Ana cone from the Bunker Hill Dike, following approximately the course of the river. The geological features are represented diagrammatically from a large number of well-logs.

Geologically speaking, according to investigations by Professor Robert T. Hill, the San Bernardino Basin is the result of block faulting. It is a sunken valley, the fill of which consists of the alluvial deposits of flowing streams. It is probable, however, that at intervals temporary lake conditions existed, as we often find them in the arid West, and this may partly explain the origin of the large lenses or layers of clay which play a prominent part in the formation of Artesian conditions. The large clay blankets or lenses apparently do not extend above the 1 150-ft. contour, and, under normal conditions, this contour seems to define the area within which Artesian flow can be produced at any point. Above this contour, clay strata seem to be isolated and of no great thickness, so that seepage waters sink without being deflected laterally. In 1904, at the end of a period of deficient rains, the water-plane above the Artesian rim had followed the line marked A-A_1. With copious rains during the last 10 years, and as a result of artificial spreading of flood waters, the basin, in its easterly portion, has been refilled, and in the spring of 1915 the water-plane was in the position marked B-B_1. Excessive rains during the winter of 1915-16 forced the water-plane in the spring of 1916 to the position, C-C_1, causing an overflow at the edge of the clay strata.

This was manifested by the appearance of a series of swamps on the Santa Ana cone along the 1 150-ft. contour, as shown on Plate XIII. A map prepared in 1888 by the State Engineer of California, and testimony of settlers, indicate that similar swamps had existed as late as 1893 along the Artesian rim of the entire basin. This was toward the end of the wet period extending from 1883 to 1893. At that time no large Artesian wells had been drilled, and artificial abstractions were limited to shallow capped wells 2 and 3 in. in diameter in San Bernardino. It may be stated, therefore, that the basin, in 1893, was in its natural condition, and that it was full and overflowing. Similar conditions obtained again in the easterly part of the San Bernardino Basin in the spring of 1916.

As far as the Santa Ana cone is concerned, well observations, covering a period of several years, have shown that the seasonal fluctuations of the water-plane above the rim occur in the same manner as those of the surface of a reservoir, namely, as fluctuations of the water-plane as a whole, corresponding to seasonal replenishment and depletion. During the winter there is a rise culminating in April or May; and during

the summer there is a drop in the water-plane terminating about October and as late as December. This is borne out by the diagrams of Plate XIV, showing the fluctuations of a number of surface wells of less than 200 ft. depth. (For location, see Plate XIII.) It will be noted that all the wells reached their peaks, as well as the low points, simultaneously. There is a marked increase in the magnitude of the fluctuations with the higher elevations.

If there were no gravel fill above the rim, then the surface of the water-plane above the same would be horizontal, like that of a reservoir, in the manner indicated by the line, B-F-F_1, in Fig. 5, producing pressure on all points in the Artesian basin below. Owing to the leakage to the surface of large quantities of water in the Artesian belt, there must be a continuous down-stream movement of water in the entire subterranean basin, and as the gravel fill causes frictional resistance to percolation, the surface of the water-plane above the rim is not horizontal, but assumes an incline, as shown in the lines, A-A_1, B-B_1, or C-C_1.

There is still a transmission of pressure from this inclined water-plane on the waters confined below the Artesian rim, but the hydraulic head at any point is equivalent only to the difference of the topographic elevation between that point and the Artesian rim.

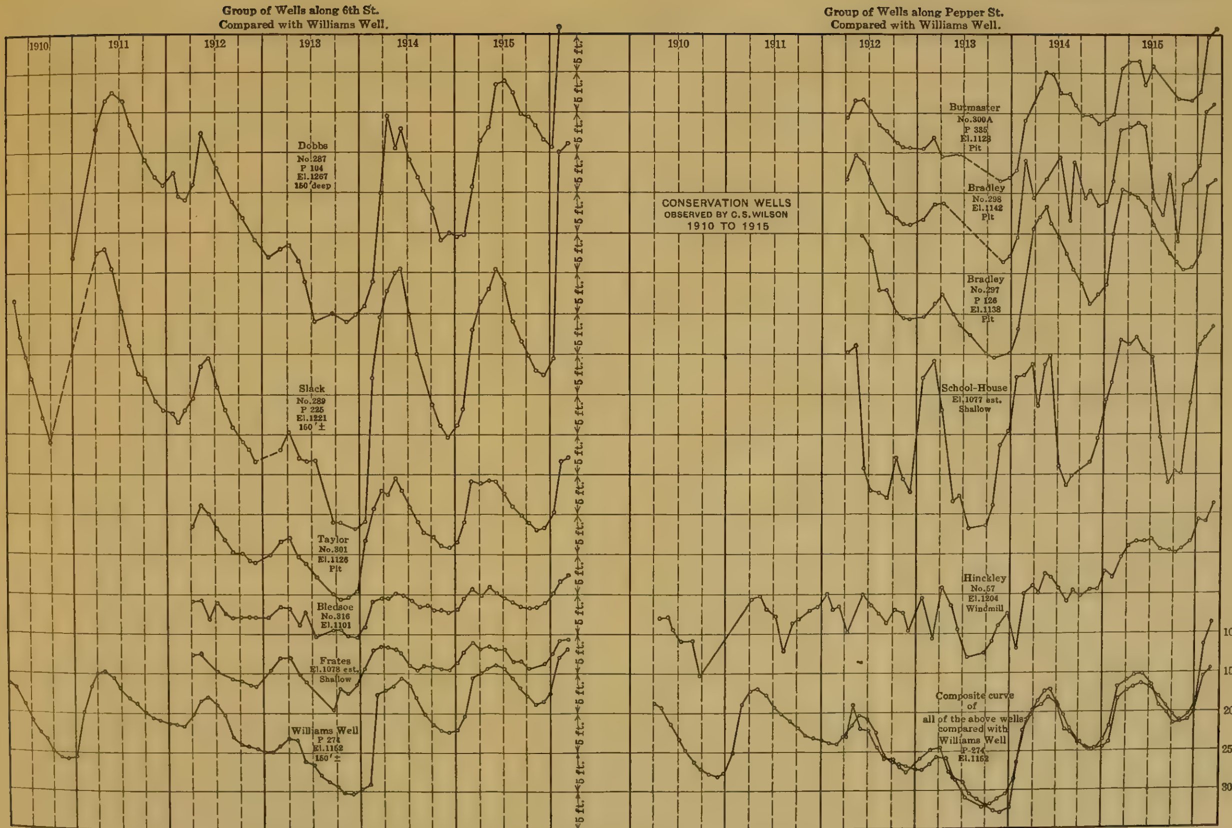
We have, therefore, two distinct phenomena: First, a transmission of pressure from the water-plane above the rim on the confined waters below the same, and second, a slow down-stream movement of the entire body of underground water.

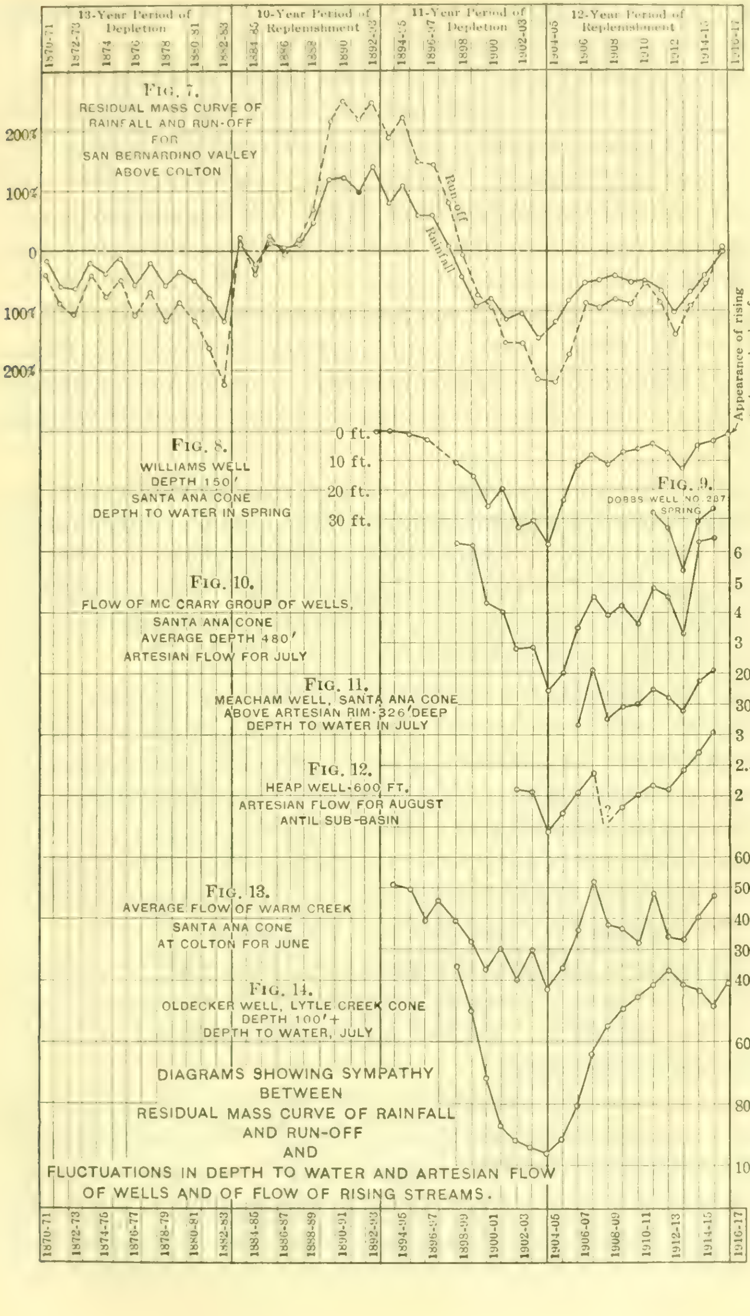
It follows, also, that there must be sympathetic action between the position of the water-plane above the rim and the pressure and discharge of Artesian wells and the flow of rising streams in the Artesian basin.

This statement is borne out by well and stream measurements made in this basin, extending over a period of 25 years. A selection of these is presented in Figs. 7 to 14, together with the residual mass curve of rainfall for San Bernardino, and the residual mass curve of run-off for all the streams tributary to the basin, expressed in percentage of the mean for a 45-year period. (See Fig. 7.) (For location of wells, see Plate XIII.)

Since 1900, ground-water fluctuations have been influenced by gradually increasing artificial abstractions and by artificial spreading of

PLATE XIV.
PAPERS, AM. SOC. C. E.
SEPTEMBER, 1917.
SONDEREGGER ON
HYDRAULIC PHENOMENA,
SAN BERNARDINO BASIN, CAL.




13-Year Period of Depletion
10-Year Period of Replenishment
11-Year Period of Depletion
12-Year Period of Replenishment
FIG. 7.
RESIDUAL MASS CURVE OF RAINFALL AND RUN-OFF FOR SAN BERNARDINO VALLEY ABOVE COLTON
Run-off
Rainfall
Appearance of rising water on banks of Santa Ana River
FIG. 8.
WILLIAMS WELL
DEPTH 150′
SANTA ANA CONE
DEPTH TO WATER IN SPRING
FIG. 9.
DOBBS WELL NO. 287
SPRING
FIG. 10.
FLOW OF MC CRARY GROUP OF WELLS,
SANTA ANA CONE
AVERAGE DEPTH 480′
ARTESIAN FLOW FOR JULY
FIG. 11.
MEACHAM WELL, SANTA ANA CONE
ABOVE ARTESIAN RIM-326′ DEEP
DEPTH TO WATER IN JULY
FIG. 12.
HEAP WELL-600 FT.
ARTESIAN FLOW FOR AUGUST
ANTIL SUB-BASIN
FIG. 13.
AVERAGE FLOW OF WARM CREEK
SANTA ANA CONE
AT COLTON FOR JUNE
FIG. 14.
OLDECKER WELL, LYTLE CREEK CONE
DEPTH 100′+
DEPTH TO WATER, JULY
Second-Feet
Feet
DIAGRAMS SHOWING SYMPATHY BETWEEN RESIDUAL MASS CURVE OF RAINFALL AND RUN-OFF AND FLUCTUATIONS IN DEPTH TO WATER AND ARTESIAN FLOW OF WELLS AND OF FLOW OF RISING STREAMS.

flood waters on the Santa Ana débris cone. (See Figs. 16 to 23, and referred to in the section following.)

Beginning with Fig. 8: This shows the highest, or spring elevation, for a period of 25 years, of the water surface in the "Williams well" of the Gage Canal Company, 6 in. in diameter, depth approximately 150 ft.; it is at an elevation of 1 152 ft., and is about 600 ft. south of the present stream bed of the Santa Ana River. The diagram may be considered as representative of the fluctuations of the water-plane near the Artesian rim and above it.

Fig. 9 gives the fluctuations for the past 5 years in the "Dobbs well"; depth, 150 ft., more or less; it is near the course of Plunge Creek, a tributary of the Santa Ana River. The elevation of the well is 1 267 ft. Its fluctuations coincide with those of the "Williams well."

Fig. 10 gives the Artesian flow of a group of five wells called the "McCrary wells", ranging in depth from 350 to 600 ft., on the banks of Warm Creek where it crosses Baseline Road; elevation, 1 093 ft. Slight deviations of fluctuations from those of the "Williams well" are explained by sympathetic action with a number of Artesian wells and pumping plants in the immediate vicinity.

Fig. 11 shows the spring elevation of the water surface of the "Meacham well", depth 326 ft., which is ¾ mile north of the edge of the Santa Ana cone, and about 1½ miles northeast of San Bernardino; surface elevation, 1 126 ft. The well was originally Artesian, the water rising to an elevation of 1 146 ft. This well is in sympathy with the deep wells of San Bernardino and the Riverside Water Company in the "Antil" sub-basin, about 1 mile east of San Bernardino; this well has been called the barometer of this sub-basin.

Fig. 12 shows the flow for August in the "Heap well" of the Riverside Water Company, situated in the "Antil" sub-basin, at an elevation of about 1 062 ft. The flow of this well is affected by that of other wells in the vicinity.

Fig. 13 shows the natural flow of Warm Creek at the Riverside Water Company's intake near Colton, for June. This discharge represents rising water, as much as 90% of which comes from the Santa Ana cone. The flow of Warm Creek is representative of the volume of natural rising water of the Santa Ana cone. (However, large volumes of water rise also in the channel of the Santa Ana River.)

The parallelism in the fluctuations of the flow of Warm Creek with those of the flow of Artesian wells, and with the water surface in surface wells above the rim, is demonstrated. The observations depicted on Figs. 8 and 13 extend over a complete cycle of dry and wet years; they indicate the return, in the spring of 1916, of conditions in the Santa Ana cone similar to those of 1893, and present a proof of the return of the water-plane to its former highest position.

In other words, the artificial draft on the Santa Ana cone has not exceeded the replenishment as it has occurred for the past 23 years, and the basin, therefore, has not been overdrawn.

The intimate relation between the volume of rainfall and run-off of the tributary streams on one side, and the fluctuations in the water-plane and Artesian flow on the other side, is easily discerned by comparison of Fig. 7 with Figs. 8 to 13.

Admitting that in 1915-16 the water-plane in the eastern portion of the San Bernardino Basin has returned to its original position of 1893—and this in the face of a very material artificial draft—then the residual mass curve of rainfall and run-off apparently does present a criterion by which the true status of the underground supply can be determined with reference to a long period of years. Without the records which are now at hand, the position of the water-plane in 1904 might have appeared exceedingly alarming, though the information which we have to-day shows that this condition was in the natural order of things, and that recuperation did follow in due time.

Fig. 14 represents the fluctuations of the water-plane in the "Oldecker well", which is 1½ miles northwest of San Bernardino, at an elevation of 1 185 ft., and outside the limits of the original Artesian belt. This well belongs to the Lytle Creek cone. Until 1911 its fluctuations, broadly speaking, corresponded to those of the residual mass curve, and to those of the other wells. In 1911-12, however, there was still a rise in the water level, which might indicate a lagging of this well. From 1912 to 1915 there was a continuous decline of the water-plane in the face of the 2 years of excessive rains of 1913 to 1915. The inference is that there was either deficient replenishment in the Lytle Creek cone after 1912, due to the diversion of flood waters at the mouth of the canyon of Lytle Creek, or that there was an excessive draft. Observations in this respect seem to point to a combination of these circumstances.

Fig. 14 tends to refute any theory which would establish the San Bernardino Basin as a unit in which there is a perfect transmission of pressure from one end of the basin to the other. It has been observed that in San Bernardino, and in the Lytle Creek cone to the west thereof, water levels in 1915-16 had not returned to their original elevations of the early Nineties, and the swamps which were characteristic of the border line of the original Artesian basin, had not returned in the Lytle Creek region.

Some observations of surface wells have been made at intervals from 1900 to 1916 by the U. S. Geological Survey. These have been used to make up Fig. 6, showing profiles of the water-plane of the Santa Ana cone for 1900, 1904, 1915, and 1916. The observations in the fall of the year present the lowest water levels for the season. There are no continuous records of pressure of any Artesian well, but for 1915 and 1916 the pressure is given for the Kehl flume well, Heap well, and McCrary wells, together with the hydraulic gradient of the water-plane of those 2 years. The friction head between the Artesian rim and the Kehl flume well for the spring of 1915 was approximately 2 ft. per thousand, and was caused by the natural leakage which appears in rising streams and swamps, as, at the time the measurements were made, but few Artesian wells were open.

Fig. 6 is in support of the general hydraulic theory which has been developed in the discussion of Fig. 5.

Another point to be brought out by Fig. 6 is that a piling up of water on the débris cone has occurred for the past 12 years. Storage in the débris cone is practicable up to the point where the basin begins to overflow over the clay blankets, as indicated by the formation of swamps.

Spreading of Flood Water.

Seepage water, if applied on the cone more or less uniformly from the mouth of the canyon to the rim, or in the upper part of the cone, would produce a simultaneous rise of the entire plane comparable to the change from the position, B-B_1 to C-C_1, Fig. 5, and the application of seepage water on the area just above the rim would produce a plane as shown by the line, C-D-B_1.

Conservation of flood water by spreading has been accomplished mainly on the cone of the Santa Ana River proper, which, with its tributary, Mill Creek, controls a water-shed of 250 sq. miles. It was

first practised by the Gage Canal Company as early as 1900, during the dry period, and a few years later by the Riverside Water Company. The method consisted of plowing the stream bed for some distance above the Artesian rim, thereby preventing a sealing up of its pores, and at the same time splitting up the stream. The effect was a change of the water-plane, similar to the one shown in Fig. 5, from the position, A-A_1 to B-F-A_1, or from B-B_1 to C-D-B_1, producing an immediate heavy efflux of rising water in the channel of the Santa Ana River along the lines, *A-B* or *B-C*. As storage was limited to comparatively small areas situated like the area, *A-F-B* or *B-C-D*, large portions of the water must have escaped almost as fast as they were put under ground, so that the effect of storage was only temporary. However, the Gage Canal Company enjoyed annually a material increase in the flow of the river during the early part of the summer.

The farther up the spreading is practised the more general will be the benefit on the entire Artesian basin. In 1906 the Gage Canal Company moved its spreading works into the vicinity of the Santa Fe Railway crossing, and in 1908 the Riverside Water Company began to spread water in the upper parts of the cone. Their efforts were united in 1909. In that year an organization of citizens representing these two companies, and other local water companies, was perfected and incorporated under the name of "Water Conservation Association."

Through the efforts of this Association, the Federal Government, by legislative act, set aside certain public lands in the Santa Ana cone to be used as spreading grounds. The lands were of no practical value for agricultural purposes, being covered with old river washes and partly overgrown with desert brush. The Act of Congress was under date of February 20th, 1909.*

Effective work has been carried on by the Association since 1909, there being now 2 640 acres under its control. (For location, see Plate XIII.) It has filed upon 500 sec-ft. of flood water of the Santa Ana River. A permanent concrete lodging house for laborers has been constructed on the grounds, and three concrete diversion weirs have been built, of a total capacity of 200 sec-ft. The maximum quantity of water diverted during any one day has been about 170 sec-ft. The center of the spreading works is about 4 miles above the 1 150-ft. contour.

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* Statute No. 36, p. 641.

Method of Diverting.

There is no permanent dam on the Santa Ana débris cone for the diversion of storm waters, and this portion of the work has always been the most difficult. The river carries great quantities of débris during floods, with boulders many tons in weight. The construction of a permanent dam on the débris cone would sooner or later cause a material change in the course of the river, and might lead to extensive damages in the valley below. Therefore, only temporary dams are built, which are washed out with every high flood. In 1915, at the suggestion of Mr. W. E. Pedley, of Riverside, a boulder dam was constructed, from 4 to 5 ft. wide on top, with a vertical up-stream slope, and a 1 to 1 down-stream slope; height, from 3 to 6 ft.; length, about 200 ft.; and enclosed with hog-wire mesh of No. 9 and No. 11 gauge. The dam was washed out during the same winter. A similar construction was used during the winter of 1916, special care being taken to protect the down-stream toe with large boulders, as a result of which the dam withstood the 1916 flood; but a by-pass, about 200 ft. in length, was washed out around the western abutment. The dam cost $581 for labor, plus $40 royalty to Mr. Pedley. The writer believes that such dams will withstand moderate floods and fulfill the purpose for which they are intended. Brush and leaves made the dam fairly tight, approximately 2 sec-ft. seeping away. However, the logical location for diversion dams is at the mouth of the canyon, where permanent structures can be maintained.

Method of Spreading.

Three methods of spreading are practised. First, from the main diversion ditch, small streams of 50 in. or less are diverted and split up into furrows, plowed by teams or made by hand. As long as no cutting sets in in the furrows, the rate of absorption is quite satisfactory, and the rivulets are not disturbed. During 1915 the maximum wetted area covered by ditches and furrows at any one time did not exceed 50 acres, and diversions varied from 120 to 170 sec-ft. The rate of absorption observed in experiments was 3.42 sec-ft. per acre of wetted area.

The second method consists of building boulder dams in the old channels of the débris cone, forming small reservoirs or ponds. They

are built from 6 to 10 ft. high, and made more or less water-tight by throwing earth against the up-stream face. Nine ponds were constructed, covering a total area of about 2 acres. The ponds, at the beginning of the season, absorb from 2 to 2.3 sec-ft. each; but their absorptive capacity decreases as the bottom becomes silted up, and at the end of the season they have to be scraped. A third method, tried as an experiment, consisted in the construction of a 5 by 5-ft. timbered pit, 40 ft. deep; cost $483. Only clear water was admitted, and the absorption did not exceed 0.7 sec-ft. constant flow.

The method of spreading by splitting up the stream into furrows is probably the most efficient, although ponds, once constructed, prove quite satisfactory, requiring but little attention. From January 1st to May 31st, 1915, one foreman, two men, and one team, were employed in spreading from 120 to 170 sec-ft. The total volume of water diverted was 26 527 acre-ft., and the total expenditure, $2 600.

Along the spreading ditches and furrows desert brush begins to die off. No water is spread during the first day or two of a storm, when the water is muddy.

It has been claimed that large quantities of water spread will evaporate. However, this is not substantiated by facts. The evaporation from still water in Southern California varies from 60 to 72 in. in depth per year, and in swamps it is approximately 96 in. per year. The percentage of evaporation during the winter, from January to May 31st, is less than one-third of the total for the year. For a wetted area of 50 acres, as observed in 1915, and assuming evaporation from spreading grounds equivalent to that from swamps, the total evaporation would not exceed 133.3 acre-ft., which, compared with the total of 26 527 acre-ft. of water spread, would be insignificant.

In order to determine the volume of net artificial absorption, it is necessary to ascertain the natural absorption which would occur in the river bed for a given stream discharge. Table 2 shows the percentage of absorption measured in the stream bed between various stations, for the season, 1914-15. (For location of stations, see Plate XIII.) The center of natural absorption is about 2 miles above the 1 150-ft. contour, that is, in the vicinity of Orange Street.

Fig. 15 shows the natural absorption curve for the Santa Ana River above the Artesian rim.' It presents averages for the rainy season of 1915, and is the result of a great number of observations made under

varying conditions, and with widely differing results. Stream gaugings were made by wading, and were limited to discharges of less than 200 sec-ft. Generally speaking, the rate of absorption is much greater after heavy storms, when the stream bed has been torn up; a prolonged period of constant flow tends to silt up the pores of the channel. The percentage of absorption decreases with increased flow, because of greater velocities. On the other hand, increased depth of water and increased pressure would tend to produce greater percolation.

TABLE 2.—AVERAGE PERCENTAGE OF ABSORPTION OF FLOW OF SANTA ANA RIVER FROM THE JUNCTION WITH MILL CREEK TO THE ARTESIAN RIM NEAR PALM AVENUE.

	Absorption percentage of flow at upper station.	Absorption percentage per mile.
For a flow of 150 sec-ft. at the junction of Santa Ana River and Mill Creek the loss is as follows:		
From junction to Station 2—⅛ mile	4	8
" Station 2 to Station 3—0.8 mile	9.5	12
" " 3 " " 4—¾ mile	16	21.3
" " 4 " " 5—Orange St. 1½ miles	30	20
" " 5 " " 6—1 mile	30	30
" " 6 to point of rising water, ½ mile (uncertain)	5	10 (?)

Assuming now a stream discharge of 200 sec-ft., of which 150 sec-ft. are to be diverted and spread, 50 sec-ft. remaining in the stream: According to the absorption curve of Fig. 15, of the flow of 50 sec-ft., 22 sec-ft. are absorbed in the natural stream channel. The total absorption accomplished, therefore, is $150 + 22 = 172$ sec-ft. The probable natural absorption for a discharge of 200 sec-ft. would have been 108 sec-ft., leaving 64 sec-ft., or $42\frac{1}{2}\%$, as net artificial absorption. On the basis of similar computations for daily flow and diversions, Table 3 has been made up, which shows the total net artificial absorption accomplished from 1912 to 1915. The average is 48.7%, or practically 50% of the quantity of water spread.

During years of low run-off the percentage of net absorption is necessarily less, as the natural absorption is comparatively large, and but small quantities escape. On the other hand, during wet winters, in excess of 60% of the water spread presents net artificial absorption.

The benefits from spreading are two-fold: First, there is the above mentioned increase in the volume of water which reaches the under-

ground basin; second, on account of the application of spread waters about 2 miles higher up on the débris cone, as compared with the location of the main natural seepage, there is an increase in the length of time for which the underflow is stored, and this applies, not only to the volume of net artificial absorption, but to the total water put under ground.

The result is a delay in the time of efflux of the water which is applied at greater distance above the rim. Some light is thrown on

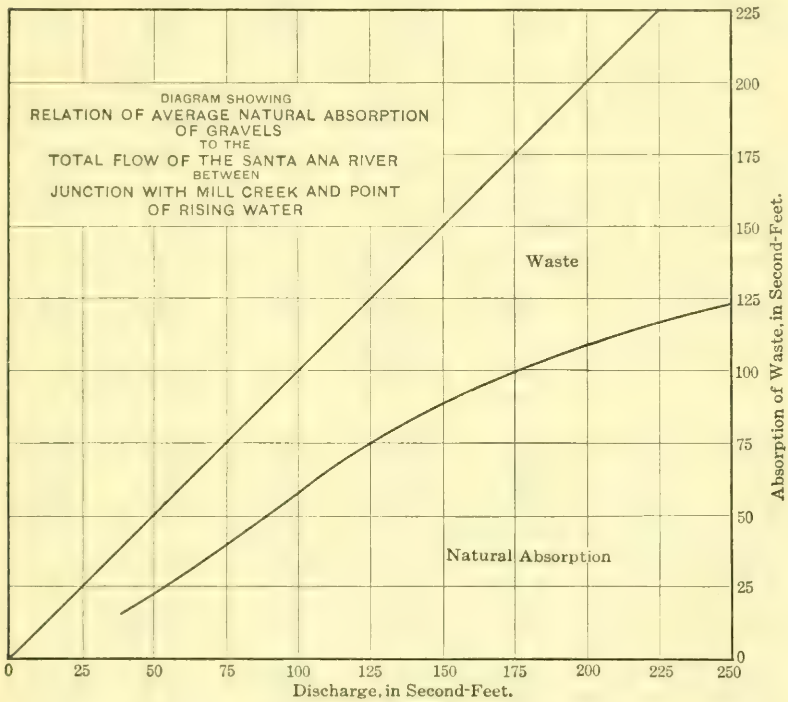


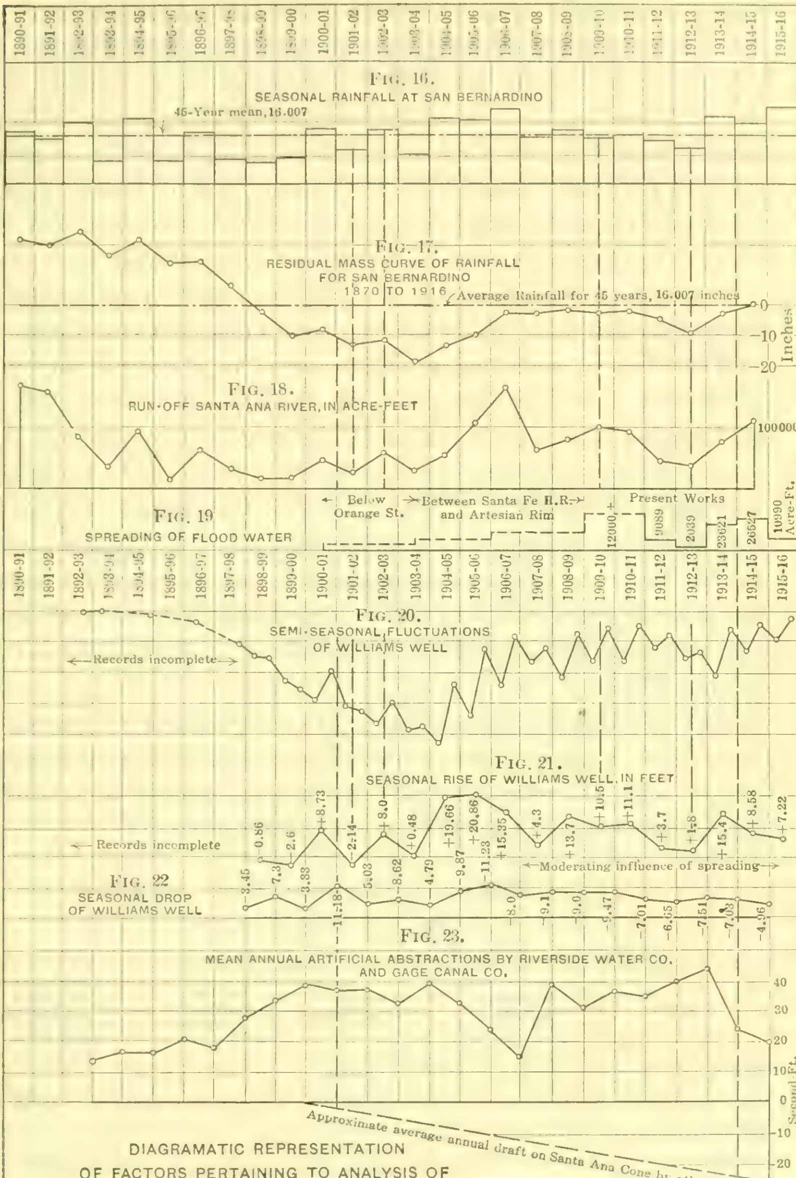
FIG. 15.

the subject by a study of the fluctuations of the "Williams well", sufficiently frequent observations of which are available, from 1898 to date. See Figs. 16 to 23. The well is 600 ft. south of the present channel of the Santa Ana River, at an elevation of 1 152 ft., and, therefore, near the Artesian rim when the basin is full. In Fig. 20 the semi-seasonal fluctuations of this well are shown, together with other data which bear on the subject.

TABLE 3.—CONSERVATION ACCOMPLISHED IN SANTA ANA CONE FROM 1912 TO 1915.

Date.	Total quantity spread, in acre-feet.	Total absorption accomplished, in acre-feet.	Computed natural absorption in river bed without spreading, in acre-feet.	NET ARTIFICIAL ABSORPTION DUE TO SPREADING.	
				In acre-feet.	Percentage.
1912					
March	4 405	4 802.0	2 875	1 927.0	44.0
April	4 176	4 880.0	3 186	1 694.0	40.5
May	508	1 130.0	834	296.0	58.0
Total for 1911-12	9 089	10 812.0	6 895	3 917.0	....
1913					
February	615	1 004.0	709	295.0	48.0
March	1 424	2 062.7	1 047	1 015.7	71.0
Total for 1912-13	2 039	3 066.7	1 756	1 310.7	....
1913					
November	115	370.0	370	00.0	0.0
December	215	673.2	601	72.2	34.0
	330	1 043.2	971	72.2	....
1914					
January	3 970	5 881.0	3 885	1 996.0	51.3
February	4 745	7 784.0	5 576	2 208.0	46.7
March	7 462	7 827.0	5 123	2 704.0	36.2
April	1 900	4 410.5	3 780	630.5	33.0
May	5 544	6 078.0	4 000	2 078.0	37.5
Total for 1913-14	23 621	31 980.5	22 364	9 616.5	....
1914					
December	713	1 222.0	747	475.0	67.0
1915					
January	1 213	1 957.0	1 186	771.0	64.0
February	4 580	9 235.0	7 220	2 015.0	44.0
March	7 800	9 668.0	6 151	3 517.0	45.0
April	4 014	8 993.0	6 663	2 330.0	58.0
May	8 920	14 624.0	8 580	6 044.0	67.0
Total for 1914-15	27 240	45 699.0	30 547	15 152.0	....
	Average net absorption				48.7

The run-off of the Santa Ana River is representative of the stream flow which, in part, produces the natural replenishment of the underground basin. The artificial abstractions by the Gage Canal



DIAGRAMATIC REPRESENTATION
OF FACTORS PERTAINING TO ANALYSIS OF
FLUCTUATIONS OF WATERPLANE OF SANTA ANA CONE
SAN BERNARDINO BASIN

Company and the Riverside Water Company are representative of fluctuations of draft on the basin. Both these companies utilize the flow of rising streams, which they supplement by the output from wells. There has been a steady increase in abstractions by other parties, notably by the City of San Bernardino, the City of Riverside, the Riverside-Highland Water Company, and by farmers operating within the limits of the Santa Ana cone. In the absence of systematic measurements, these abstractions are indicated by a dash line; in a moderated form they are subject to fluctuations similar to those of the Riverside Water Company and the Gage Canal Company.

The effect of spreading can be observed in a general way in a moderation of the fluctuations in the "Williams well" from 1907 to 1916. Other conditions being equal, the magnitude of seasonal fluctuations of the water-plane depends on the position of the plane, that is, whether the basin is full or in a state of depletion. The higher the water-plane the greater will be the pressure on the Artesian strata and the larger the volume of water escaping, and, therefore, the greater the seasonal drop; and *vice versa*. On the other hand, with equal replenishment, the seasonal rise is greater for a depleted plane than for a full basin.

This phenomenon is shown in detail in the 6th and 7th profiles of Plate XIV. Comparing seasons where conditions of rainfall and run-off are alike, the effect of artificial spreading should be noticeable by a greater seasonal rise. This is shown in Table 4.

TABLE 4.—SEASONAL RISE.

(Figs. 16 to 23.)

Season.	Rainfall, in inches.	Run-off, in acre-feet.	Seasonal rise, Williams well, in feet.	Additional rise attributed to spreading, in feet.
1902-03............	17.42	59 600	+ 8.0	+ 2.5
1909-10............	15.02	58 000	+ 10.5	
1901-02............	11.15	26 200	+ 0.48	+ 1.3
1912-13............	11.08	37 600	+ 1.8	

A still better criterion is represented by the seasonal drop, as shown in Fig. 22. It will be noticed that after 1907, and up to 1911, the drop became more uniform, varying from 8 to 9.5 ft. From 1911 to 1915 it has fluctuated between 6.5 and 7.5 ft. Prior to 1907 a higher water-plane generally caused a greater seasonal drop, but, subsequent to 1907, this rule does not obtain, as is shown by Table 5.

TABLE 5.—SEASONAL DROP.

(Fig. 22.)

Season.	Rainfall, in inches.	Depth to water, Williams well, in feet.	Seasonal drop, in feet.	Reduction of seasonal drop attributed to spreading, in feet.
1900–01...........	17.36	20	— 11.18	....
1902–03...........	17.42	30	— 8.62	....
1910–11...........	16.40	4.5	— 7.01	1.61 to 4.17
1913–14...........	21.45	5.5	— 7.03	1.58 to 4.15

It may be stated, therefore, that the effect of spreading has been to increase the seasonal rise and decrease the seasonal drop of the water-plane above the Artesian rim. Apparently, the higher up on the débris cone the spreading process is practised the longer will be the time of storage and the greater the benefit.

A reduction of the seasonal drop of from 2 to 3 ft. at the Artesian rim is equivalent to an average of from 3 to 4 ft. for the entire basin of the Santa Ana cone above the rim. This means that, as a result of spreading, a prism of valley fill of a base area equivalent to the area of the Santa Ana cone above the rim, and of an average height of from 3 to 4 ft., has been annually preserved in a saturated condition. The area referred to is approximately 15 000 acres, and the volume of the prism is from 45 000 to 60 000 acre-ft. Allowing one-third for voids in the gravel fill, the annual beneficial storage would be not less than 15 000 acre-ft. The value of 1 acre-ft. of water in the San Bernardino country varies from \$5 to \$8, and the annual benefit due to spreading operations may be estimated at \$100 000. The expenditures for spreading have varied from \$1 000 to \$3 000 per year, and the cost per acre-foot of water put under ground, from 5 to 20 cents.

The foregoing analysis also points to the benefits to be derived from the spreading of flood waters during years of moderate and deficient rains, when the stream bed naturally absorbs all the flow, provided, however, that the spreading is done in the upper parts of the cone.

It is of interest to note that the variations in artificial abstractions from 1911 to 1915, as depicted in Fig. 23, do not seem to affect the seasonal drop, meaning that, whatever the volume of artificial abstractions may have been during those years, it was less than the quantity of water which would naturally have escaped.

The question arises whether, in view of the 3 years of excessive rains, from 1913 to 1916, the water-plane would have returned, regardless of spreading operations. Rainfall conditions during these years were very much like those of 1904 to 1907, when spreading was done, but on a small scale. The effect during these years was a rise in the Williams well, from a depth of 42.87 ft. in the fall of 1904, to 8.1 ft. in the spring of 1907, or a total of 34.77 ft. On the other hand, in the fall of 1913—that is, at the beginning of the last 3-year period of excessive rains—the water stood only at a depth of 20.57 ft. In view of this, it is believed that the water-plane in 1916 would have returned to its original elevation of the early Nineties, regardless of spreading. It was observed that large volumes of rising water, varying from 80 to 150 sec-ft., appeared in the Santa Ana River in the spring of both 1915 and 1916, indicating that in the spring of 1915 the basin was full and beginning to overflow, and that a large percentage of the spread water must have flowed out over the Artesian rim.

This fact, however, does not impair the value of spreading operations. Undoubtedly, for a number of years prior to 1915, the water-plane was maintained at a higher elevation, and parallel therewith the flow of rising streams and wells in the valley was greatly in excess of what it naturally would have been, annually shortening thereby the time during which deficiencies in Artesian flow had to be supplemented by pumping. Furthermore, if, prior to 1915, we had entered a period of dry years, the basin would have been in better condition to tide over the same.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

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THE SUBSIDENCE OF MUCK AND PEAT SOILS IN SOUTHERN LOUISIANA AND FLORIDA

By Charles W. Okey, Assoc. M. Am. Soc. C. E.

To be Presented November 7th, 1917.

Synopsis.

The object of this paper is to call the attention of engineers to the fact that, in designing drainage improvements, it is often necessary to anticipate the subsidence of muck and peat lands subsequent to drainage.

The results of some observations made in England on the subsidence of drained muck and peat lands are reviewed, and the results of first-hand observations in Louisiana and Florida are shown in detail on profiles.

It was evident that, in small districts drained by pumps, the subsidence was not a very important feature, but that in large gravity drainage districts covered with deep muck or peat, where slopes were slight and the available fall was small, subsidence would be of sufficient magnitude to make a change in design necessary.

It has long been a matter of common knowledge that swamp lands, which have soils containing a large percentage of vegetable matter, subside when drained and cultivated. In the Fenland of England such soils have been drained and cultivated for a long period, with a consequent change in surface elevation. This change has been of

Note.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and, when finally closed, the papers, with discussion in full, will be published in *Transactions*.

such magnitude that it has made necessary the altering of a majority of the pumping plants that formerly gave adequate drainage to the lands they served. It is believed that it will be best to give the actual figures from a few representative cases rather than to state any general figures.

In the *Memoirs* of the Geological Survey of England and Wales for 1877 there appears a volume written by S. B. J. Skertchly on the "Geology of the Fenland." Mr. Skertchly mentions the general sinking of the surface on all the cultivated and drained Fens, where there is muck or peat on the surface, and gives definite figures for several locations. In 1848, an iron column, graduated in feet and inches, was sunk down through the peat, in the Middle Level of the Great Bedford Level in the vicinity of Whittlesey Mere, into the solid clay, so that the top of the column was level with the surface.* This column bears on the capital the inscription, "Level of the Ground in 1848." When Mr. Skertchly measured the subsidence of the surface on August 25th, 1870, he found it to be 7 ft. 8 in. The subsidence was measured again in 1875 and found to be 7 ft. 9 in.

Richard F. Grantham states† that in 1876 after erecting new engines and pumps, made necessary by the subsidence of the land, the total subsidence was then observed to be 8 ft. This was evidently after the new pumping plant had been put in, hence the drop of 3 in. in a year, due to the lowering of the water-table. In 1913 the total subsidence was observed by Mr. Grantham to be 10 ft. The peat at this point was originally 18 ft. thick.

Mr. Skertchly in his "Geology of the Fenland" also gives a table of the subsidence at this and other localities. This table is reproduced as Table 1.

The observers for the data in Table 1 were:

No. 1. W. H. Wheeler.

2–5. W. Wells. *Journal,* Royal Agricultural Society, Vol. XXI, 1860.

6. S. B. J. Skertchly.

7. W. Wells.

8. W. Marshall.

9. S. B. J. Skertchly.

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* *Journal,* Royal Agricultural Society, Vol. XXI, 1860.

† *Proceedings,* Institution of Mechanical Engineers, London, July, 1913.

TABLE 1.—COMPRESSION OF PEAT BY DRAINAGE

No.	Locality.	Dates.	Time, in years.	Thickness, in feet.	Total compression, in inches.	Annual compression, in inches.	Total compression. Percentage.	Annual compression. Percentage.
1	East Fen..................	1806–66	60	6	24	0.4	33.3	0.55
2	Whittlesea Mere.........	1851–60	9	18	42	4.7	19.5	2.2
3	" "	do.	9	18	59	6.6	27.3	3.0
4	" "	do.	9	18	66	7.4	30.5	3.4
5	" "	do.	9	18	73	8.1	33.8	3.7
6	" "	1848–70	22	18	92	4.18	42.6	1.9
7	" "	1848–75	27	18	93	3.44	43.0	1.59
8	Hilgay......................	....	26	10	52	5.2	43.3	1.7
9	Wood Fen.................	1854–74	20	8	37	1.85	38.5	1.9

In commenting on Table 1, Mr. Skertchly states:

"The mean annual percentage of compression from the above data is 2.2, which under similar circumstances may be taken as the rate for a long series of years. By 'similar circumstances' is meant low-lying peat in which very little fall can be obtained for the drains; for it is clear that if drains could be cut deep into or through the peat the compression would be more rapid."

Swamp soils which are largely of vegetable origin are classed as cumulose by the United States Bureau of Soils, with the subdivisions of peat, peaty muck, and muck. Peat is composed almost entirely of partly decomposed vegetable matter; muck has a considerable percentage of silt, clay, and sand mixed with well-decayed and finely-divided vegetable matter. The different types merge into each other. Although it is not certain that all the soils considered in this paper would be properly classed as muck soils, the term will be used, because it will probably fit most of the cases and because it is in common usage in Southern Louisiana and Florida. The purpose of this paper is not to classify the soils examined, but to give a recital of facts observed.

The writer has been engaged for the past 7 years in making investigations of the drainage of muck soils in Southern Louisiana and for the past 2 years has had charge of similar work in Florida. This work was carried on for the U. S. Department of Agriculture, Office of Public Roads and Rural Engineering, Division of Drainage Investigations, and was under the immediate supervision of the Division Chief, S. H. McCrory, M. Am. Soc. C. E., under the general direction of L. W. Page, M. Am. Soc. C. E., Director of the Bureau.

The subsidence of such soils is due to three main causes: drying, decay, and cultivation. Shrinkage and the consequent subsidence, due to drying, affects most the top layer, but it extends in a decreasing rate as deep as the soil is drained. Tests have shown that undrained Louisiana muck shrinks about 60% in volume when completely dry, and that it will regain only 70% of its original volume when saturated for a long time. Therefore, long-continued dry weather and deep drainage cause a shrinkage in the deeper layers of muck which is not counterbalanced by an increase in volume due to subsequent precipitation, or rising of the water-table. Only such part of the muck as is always saturated is entirely free from shrinkage due to drying.

The vegetable material in muck soils exists in a state of partial decay. In the undrained state it is saturated with water to such an extent that the process of decay is relatively slow. After drainage the air enters, and decay is much more rapid. The warm, humid climate of Southern Louisiana and Florida is very favorable to the rapid decay of vegetable material, much more so than in places where the surface is frozen for a part of the year. Like the effect of drying, that of decay is greatest in the top layer of material; but examination has shown that, after some years of drainage, the character of the muck at a depth of 2 ft. had been materially changed from its condition prior to drainage. Although the effect of decay is not as rapid in action as that due to drying, it is practically continuous. The complete decay of the vegetable material causes some loss of weight and a considerable loss in volume, thus gradually reducing the surface elevation of the muck.

As the effect of both the foregoing agencies is greatest in the top layer of the muck, the density of this top layer is gradually increased. After this comparatively dense material attains a thickness of a few inches, it prevents free circulation of the air into the muck below, and then drying and decay are much slower in their action. Eventually, this layer attains such a thickness that further subsidence of the surface is scarcely noticeable.

Cultivation increases the subsidence directly, by the mechanical effect of weight compacting the soil, and indirectly, by increasing the action of drying and decay. Muck soils which are so soft after drainage that they will not permit of the use of farm animals and machinery are compacted from 4 to 6 in. by the first plowing. This

first plowing is usually done with some form of tractor, with broad wheels which cover practically all the surface plowed. Subsequently, when the muck is cultivated with farm animals and machinery, the surface receives unit pressures far greater than those exerted by the broad-wheeled tractor, and a further compacting results. The underlying material, which is turned to the surface by plowing, is exposed to a greater drying action than would otherwise result. Decay, also, is hastened in the material thus brought to the surface. It is the experience in cultivating newly-reclaimed muck soils that, for a number of years after the first cultivation, a uniform depth of plowing will bring to the surface each year a considerable layer of muck which was undisturbed by the previous year's plowing. This layer of new material gradually decreases in thickness, as the thickness of the layer of plowed material gradually increases. Finally, the cultivated layer attains such a density that the combined forces of drying, decay, and compacting reduce its thickness very little. If the land is not plowed deeper than this layer, the subsidence of the remaining muck is very slow; but, if the land is plowed deep enough to reach undisturbed muck, further subsidence results.

The rate at which these agencies cause muck soils to subside has been uncertain, and it was with the idea of getting definite information that the writer was authorized to make comprehensive field investigations. The work was undertaken in Southern Louisiana in the spring of 1915, and in Florida in the early spring of 1916. The field work in Louisiana was done by the writer, and the work in Florida by Mr. F. E. Staebner under the direction of the writer. In all, 22 districts were examined. The investigations covered muck ranging in depth from a few inches to 16 ft.; it included undrained muck, that which had been under cultivation for 17 years, and that which had such a large percentage of vegetable material that it would perhaps properly be classed as peat; it also included other muck which contained a large percentage of river silt. Profiles were run on the drained and undrained muck. Samples of muck were taken, and the weight of the dry material, per cubic foot, was determined. An attempt has been made to place all the information gathered on the profiles, Figs. 1 to 13, in such a manner that the details for each district can thus be more readily grasped than from lengthy descriptions.

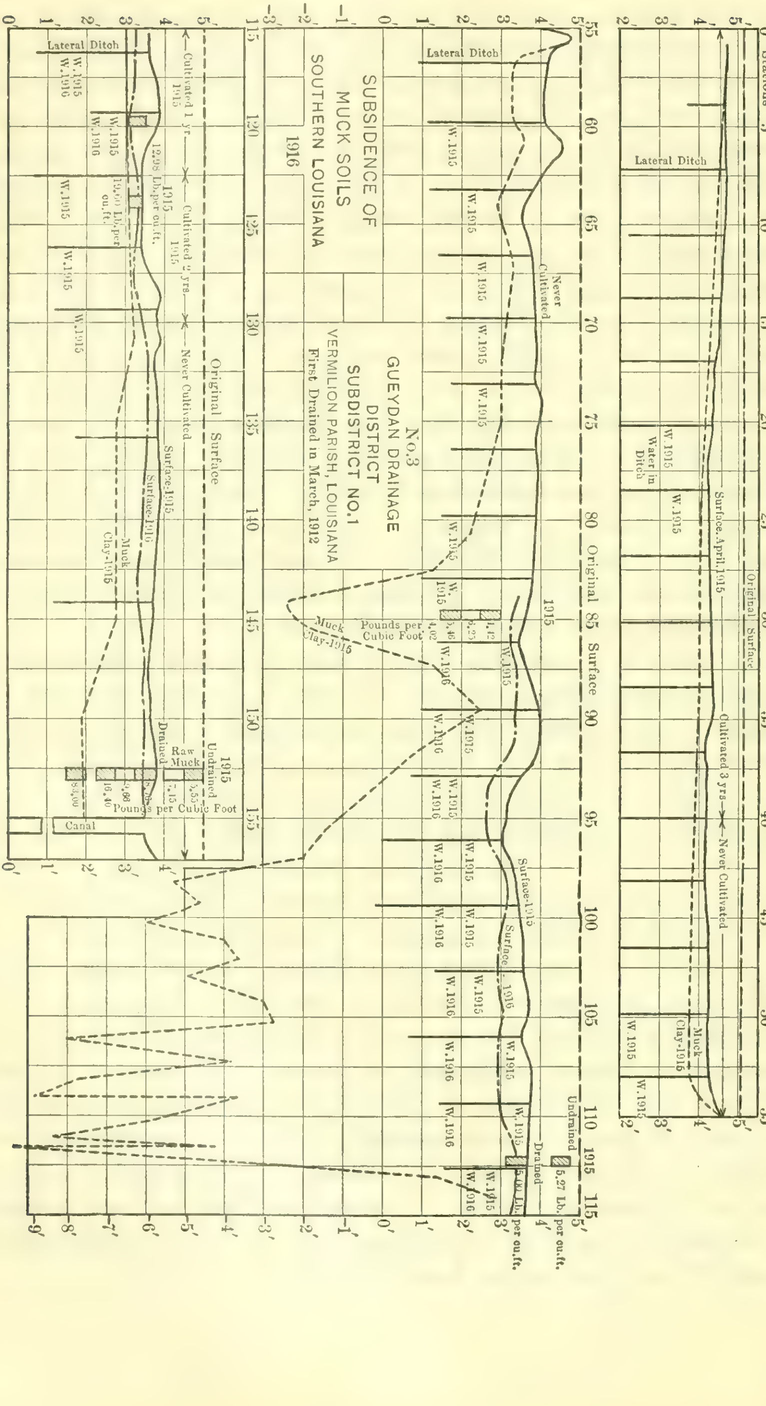

SUBSIDENCE OF MUCK SOILS
SOUTHERN LOUISIANA
1916
No.3
GUEYDAN DRAINAGE DISTRICT
SUBDISTRICT NO.1
VERMILION PARISH, LOUISIANA
First Drained in March, 1912

FIG. 1.

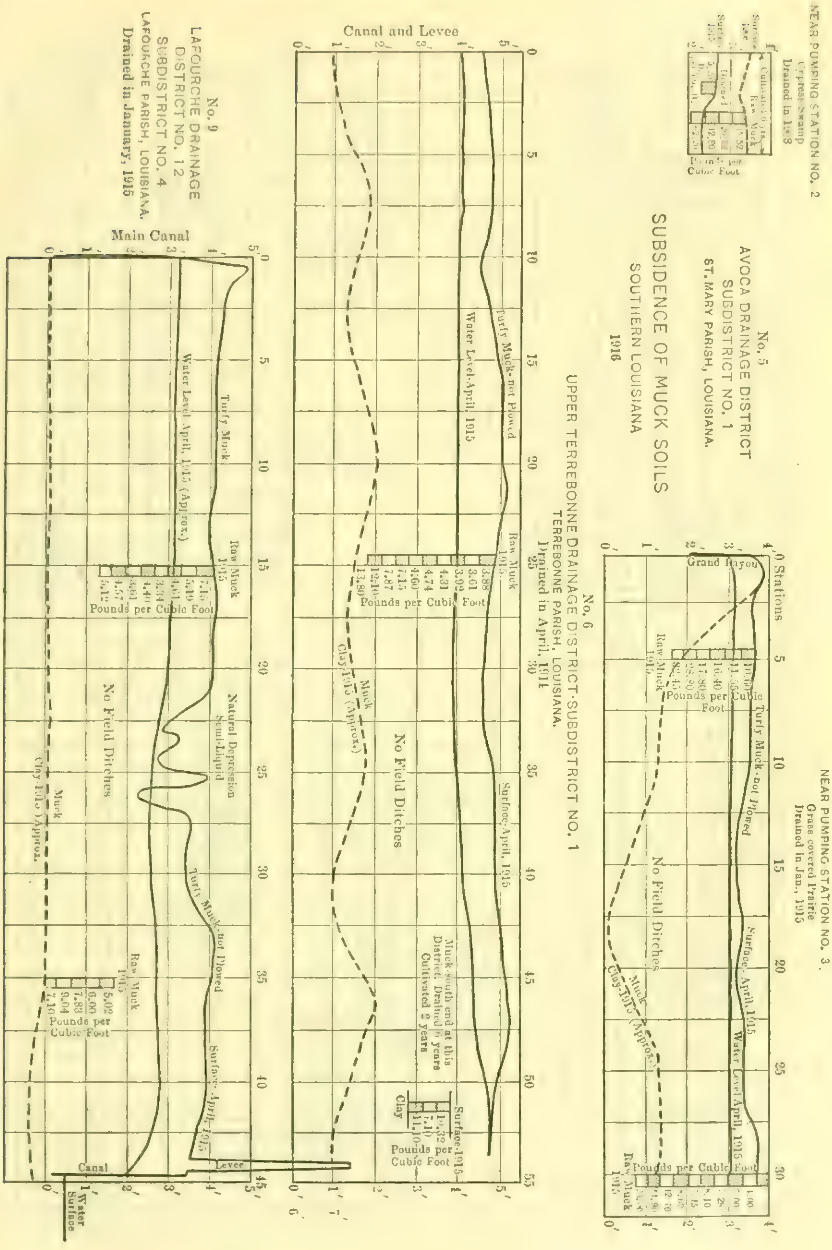

SUBSIDENCE OF MUCK SOILS
SOUTHERN LOUISIANA
1916
No. 5
AVOCA DRAINAGE DISTRICT
SUBDISTRICT NO. 1
ST. MARY PARISH, LOUISIANA.
NEAR PUMPING STATION NO. 2
Cypress Swamp
NEAR PUMPING STATION NO. 3.
Grass covered Prairie
Drained in Jan., 1915
Grand Bayou
Stations
Turfy Muck-not Plowed
Surface-April, 1915
Water Level-April, 1915
No Field Ditches
Pounds per Cubic Foot
Raw Muck 1915
No. 6
UPPER TERREBONNE DRAINAGE DISTRICT-SUBDISTRICT NO. 1
TERREBONNE PARISH, LOUISIANA.
Drained in April, 1914
Canal and Levee
Water Level-April, 1915
Clay 1915 (Approx.)
Muck
Clay
No. 9
LAFOURCHE DRAINAGE
DISTRICT NO. 12
SUBDISTRICT NO. 4
LAFOURCHE PARISH, LOUISIANA.
Drained in January, 1915
Main Canal
Turfy Muck
Natural Depression
Semi-Liquid
Canal
Levee
Water Surface

FIG. 2.

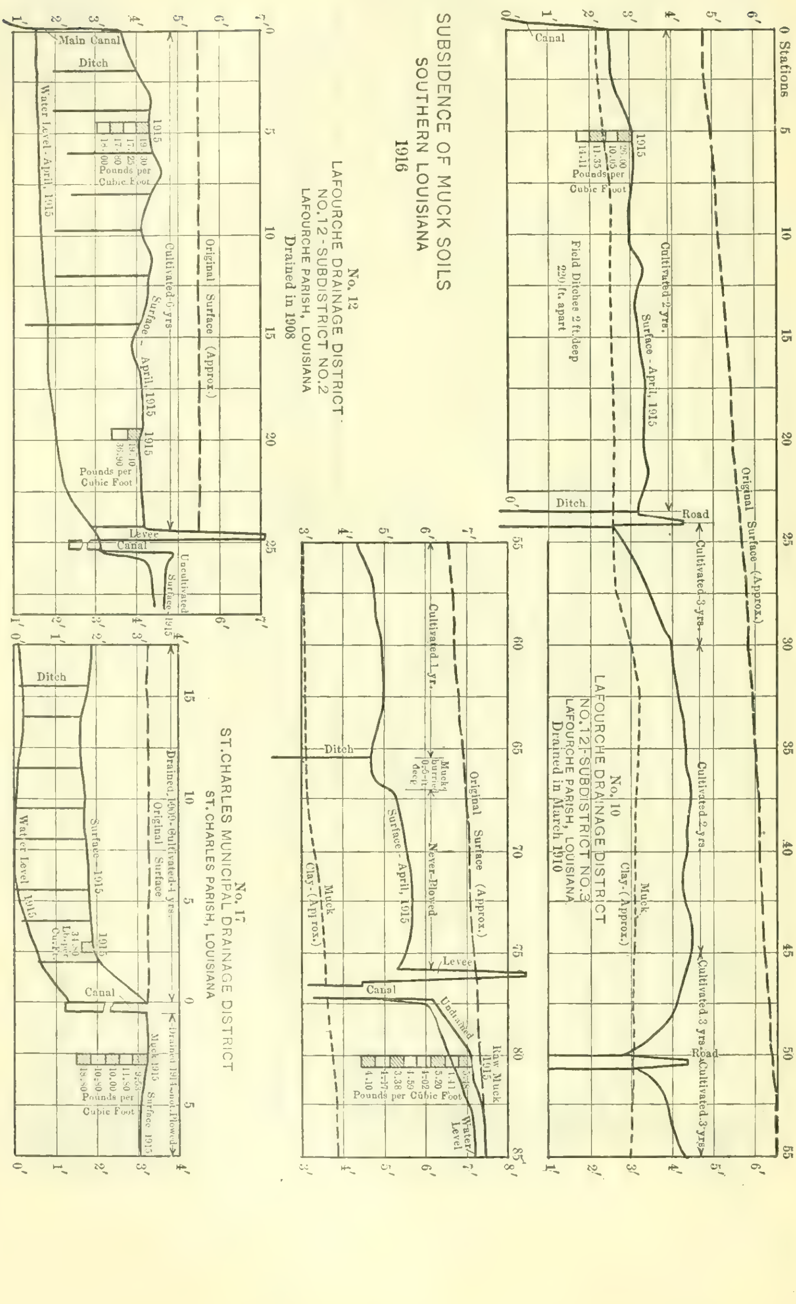

SUBSIDENCE OF MUCK SOILS
SOUTHERN LOUISIANA
1916
No. 12
LAFOURCHE DRAINAGE DISTRICT
NO. 12 - SUBDISTRICT NO. 2
LAFOURCHE PARISH, LOUISIANA
Drained in 1908
No. 10
LAFOURCHE DRAINAGE DISTRICT
NO. 12 - SUBDISTRICT NO. 3
LAFOURCHE PARISH, LOUISIANA
Drained in March 1910
No. 17
ST. CHARLES MUNICIPAL DRAINAGE DISTRICT
ST. CHARLES PARISH, LOUISIANA
Original Surface (Approx.)
Surface - April, 1915
Water Level - April, 1915
Pounds per Cubic Foot
Main Canal
Ditch
Canal
Levee
Road
Cultivated 6 yrs.
Cultivated 2 yrs.
Cultivated 3 yrs.
Cultivated 1 yr.
Never Plowed
Field Ditches 2 ft. deep 220 ft. apart
Muck
Clay - (Approx.)
Raw Muck 1915
Undrained
Uncultivated Surface 1915
Drained 1909 - Cultivated 4 yrs.
Drained 1914 - not Plowed
Surface - 1915
Water Level 1915
0 Stations

FIG. 3.

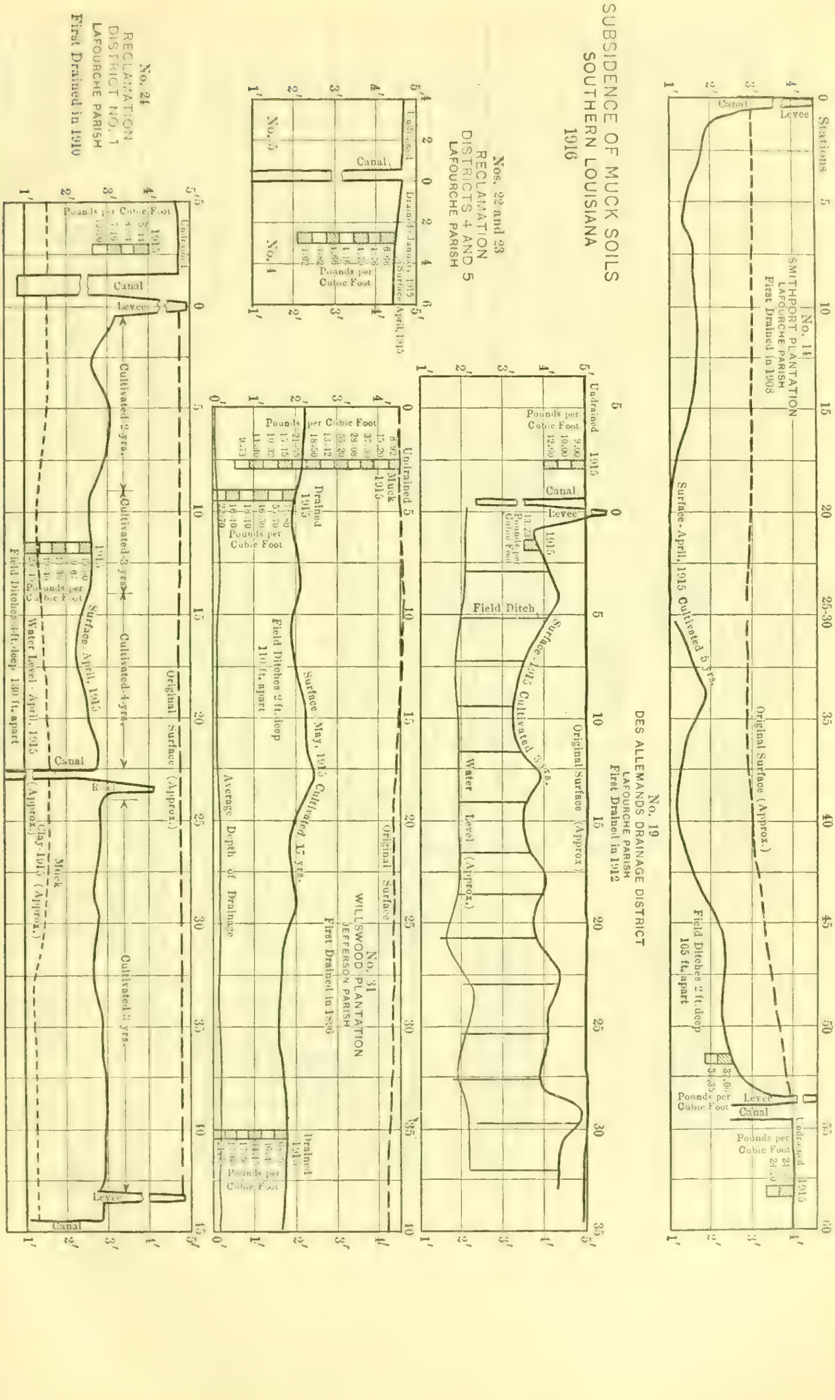

SUBSIDENCE OF MUCK SOILS
SOUTHERN LOUISIANA
1916
No. 14
SMITHPORT PLANTATION
LAFOURCHE PARISH
First Drained in 1908
Nos. 22 and 23
RECLAMATION
DISTRICTS 4 AND 5
LAFOURCHE PARISH
No. 19
DES ALLEMANDS DRAINAGE DISTRICT
LAFOURCHE PARISH
First Drained in 1912
No. 31
WILLSWOOD PLANTATION
JEFFERSON PARISH
First Drained in 1896
No. 24
RECLAMATION
DISTRICT NO. 1
LAFOURCHE PARISH
First Drained in 1910

FIG. 4.

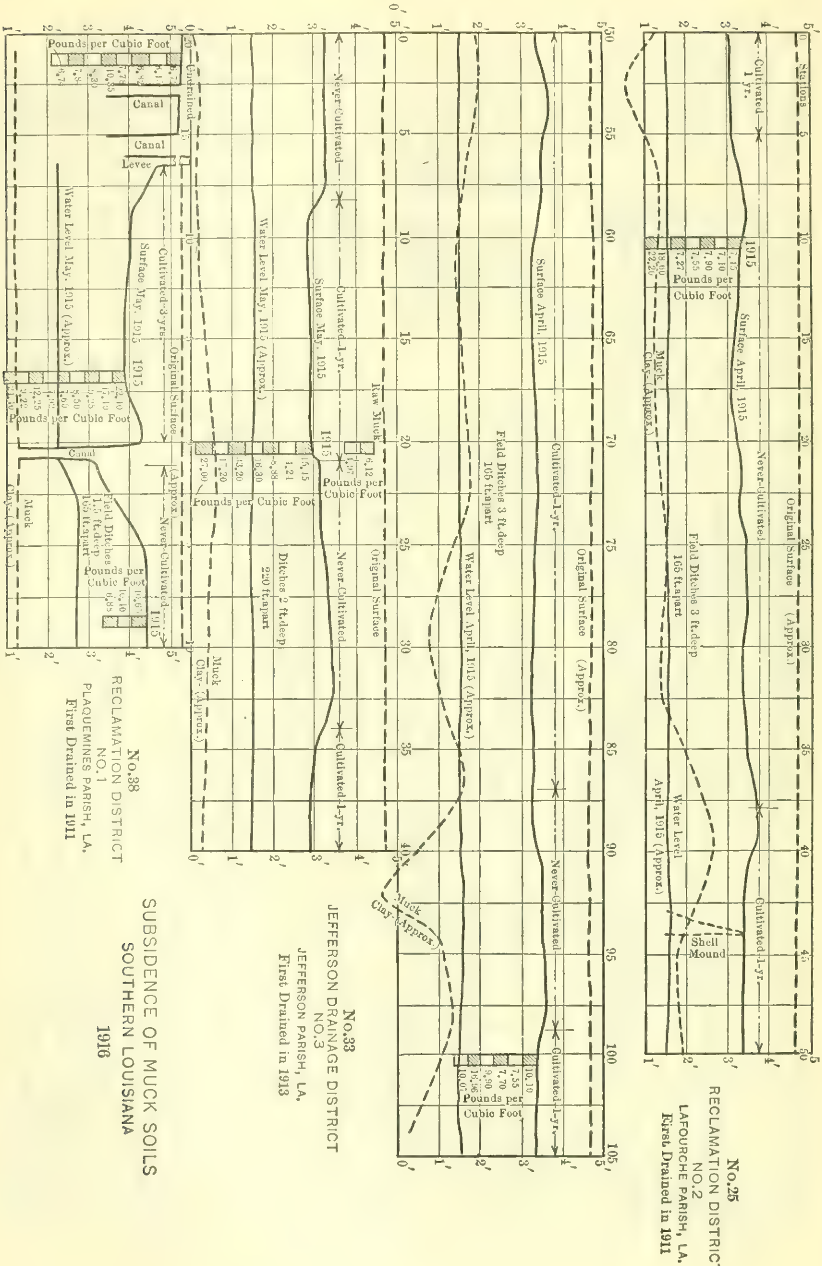
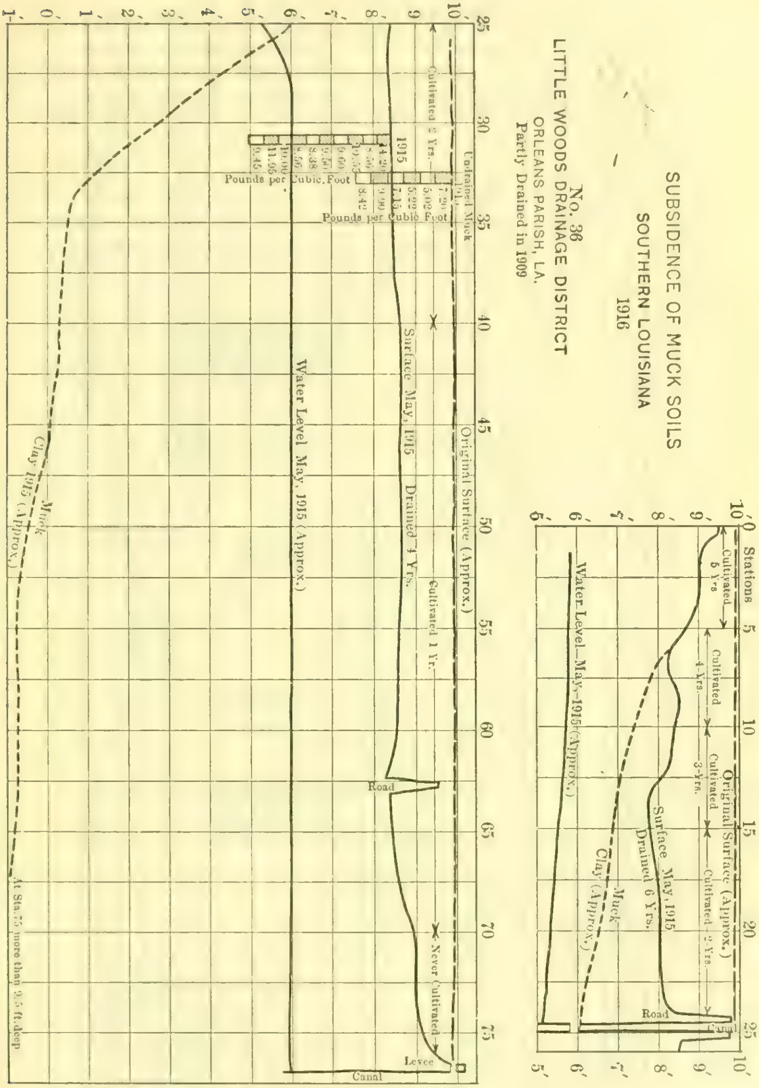

SUBSIDENCE OF MUCK SOILS
SOUTHERN LOUISIANA
1916
No.38
RECLAMATION DISTRICT
NO.1
PLAQUEMINES PARISH, LA.
First Drained in 1911
No.33
JEFFERSON DRAINAGE DISTRICT
NO.3
JEFFERSON PARISH, LA.
First Drained in 1913
No.25
RECLAMATION DISTRICT
NO.2
LAFOURCHE PARISH, LA.
First Drained in 1911
Pounds per Cubic Foot
Canal
Levee
Water Level May, 1915 (Approx.)
Surface May, 1915
Cultivated-3-yrs.
Original Surface
Never-Cultivated
Field Ditches 1.5 ft.deep 165 ft.apart
Muck
Clay-(Approx.)
Undrained
Cultivated-1-yr.
Raw Muck
Ditches 2 ft.deep 220 ft.apart
Surface April, 1915
Water Level April, 1915 (Approx.)
Field Ditches 3 ft.deep 165 ft.apart
Stations
Shell Mound

FIG. 5.



SUBSIDENCE OF MUCK SOILS
SOUTHERN LOUISIANA
1916
No. 36
LITTLE WOODS DRAINAGE DISTRICT
ORLEANS PARISH, LA.
Partly Drained in 1909
Original Surface (Approx.)
Surface May, 1915
Drained 4 Yrs.
Water Level May, 1915 (Approx.)
Cultivated 2 Yrs.
Cultivated 1 Yr.
Never Cultivated
Undrained Muck
1915
Pounds per Cubic Foot
Road
Levee
Canal
Muck
Clay 1915 (Approx.)
At Sta. 75 more than 9.5 ft. deep
Stations
Cultivated 6 Yrs.
Cultivated 4 Yrs.
Cultivated 3 Yrs.
Cultivated 2 Yrs.
Surface May, 1915
Drained 6 Yrs.
Water Level May, 1915 (Approx.)
Clay (Approx.)

FIG. 6.

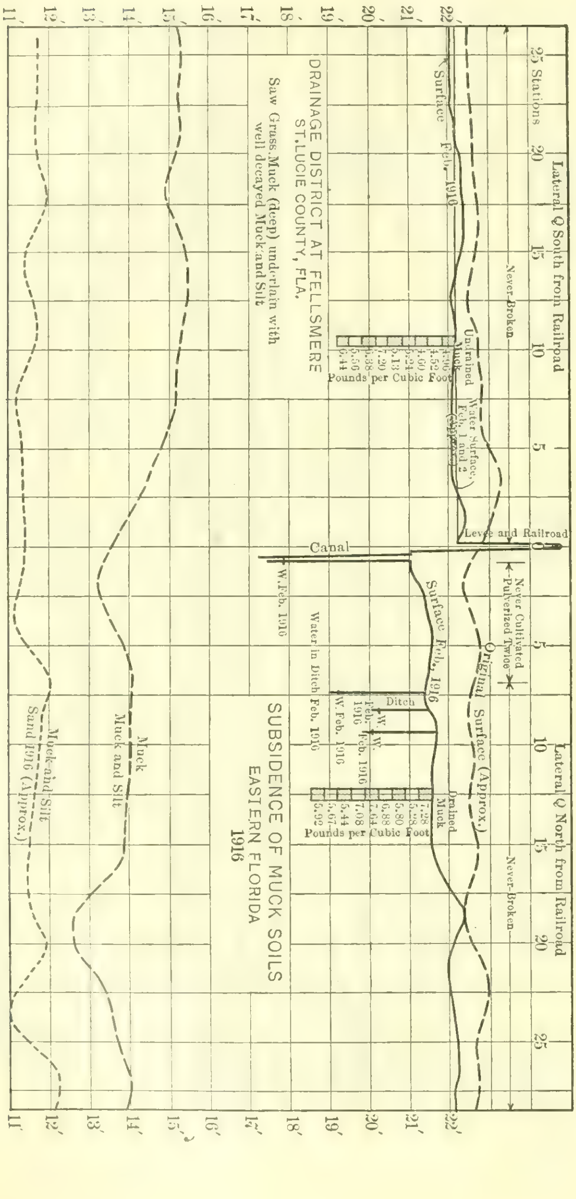

SUBSIDENCE OF MUCK SOILS
EASTERN FLORIDA
1916
DRAINAGE DISTRICT AT FELLSMERE
ST. LUCIE COUNTY, FLA.
Saw Grass Muck (deep) underlain with
well decayed Muck and Silt
Lateral ℄ South from Railroad
Lateral ℄ North from Railroad
Stations
Never-Broken
Never Cultivated
Pulverized Twice
Original Surface (Approx.)
Surface Feb., 1916
Water Surface, Feb. 1 and 2
Undrained Muck
Drained Muck
Pounds per Cubic Foot
Levee and Railroad
Canal
W. Feb. 1916
Water in Ditch Feb. 1916
Ditch
Muck
Muck and Silt
Muck and Silt
Sand 1916 (Approx.)

FIG. 7.

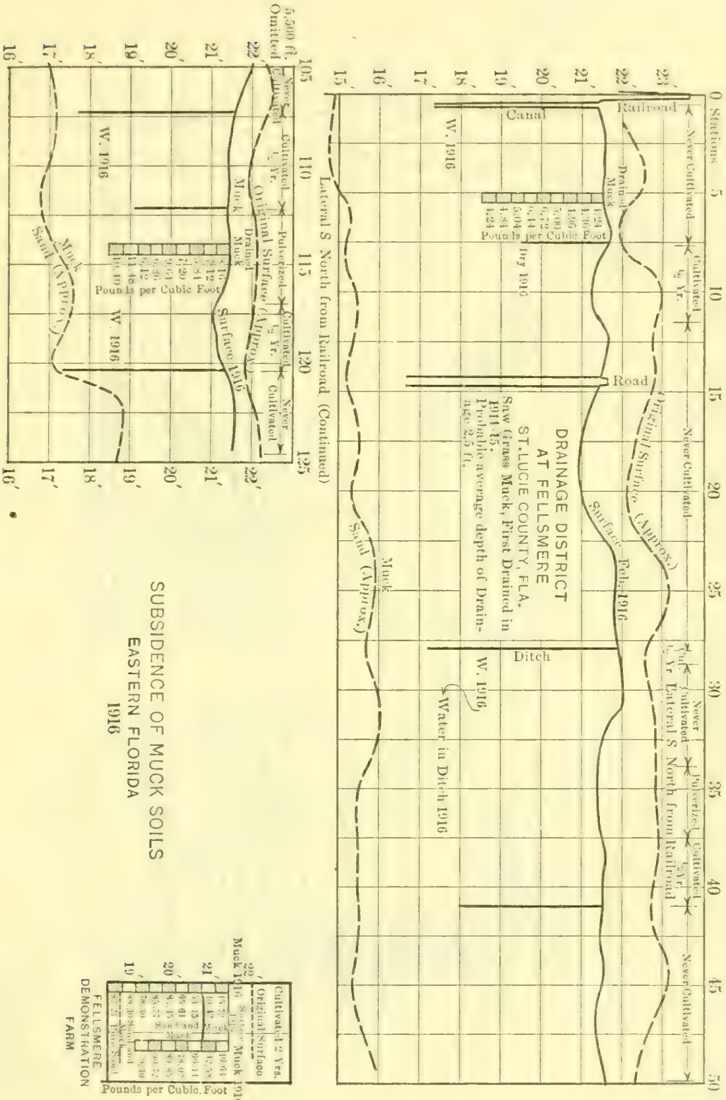

SUBSIDENCE OF MUCK SOILS
EASTERN FLORIDA
1916
DRAINAGE DISTRICT
AT FELLSMERE
ST. LUCIE COUNTY, FLA.
Saw Grass Muck, First Drained in 1914-15.
Probable average depth of Drainage 2.5 ft.
Stations
Lateral S North from Railroad
Lateral S North from Railroad (Continued)
5,500 ft. Omitted
Railroad
Canal
Road
Ditch
Never Cultivated
Cultivated ½ Yr.
Pulverized
Original Surface (Approx.)
Surface Feb. 1916
Drained Muck
Muck
Sand (Approx.)
W. 1916
Water in Ditch 1916
Dry 1916
Pounds per Cubic Foot
Surface 1916
FELLSMERE
DEMONSTRATION
FARM
Cultivated 2 Yrs.
Original Surface
Surface Muck 1916
Muck 1916
Sand and Muck
Pure Sand

FIG. 8.

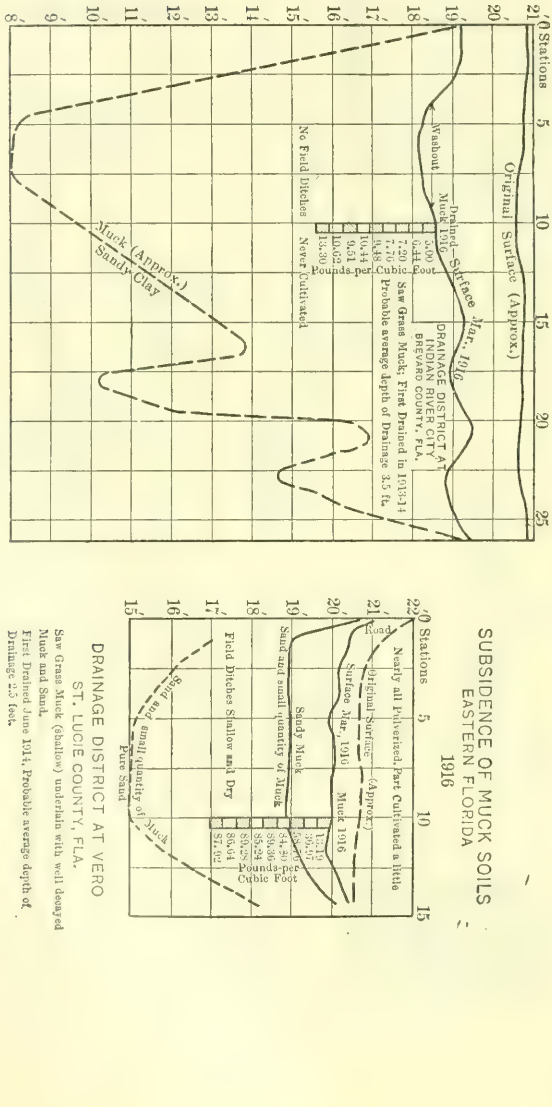

SUBSIDENCE OF MUCK SOILS
EASTERN FLORIDA
1916
Stations
Original Surface (Approx.)
Drained—Surface Mar., 1916
Muck 1916
Washout
DRAINAGE DISTRICT AT INDIAN RIVER CITY
BREVARD COUNTY, FLA.
Saw Grass Muck; First Drained in 1913-14
Probable average depth of Drainage 3.5 ft.
5.00
6.44
7.20
7.76
9.48
10.44
9.51
10.62
13.30
Pounds per Cubic Foot
No Field Ditches
Never Cultivated
Muck (Approx.)
Sandy Clay
Nearly all Pulverized. Part Cultivated a little
Road
Original Surface (Approx.)
Surface Mar., 1916
Muck 1916
Sandy Muck
Sand and small quantity of Muck
Field Ditches Shallow and Dry
Sand and small quantity of Muck
Pure Sand
Pounds-per Cubic Foot
13.19
36.97
84.30
89.30
85.24
89.28
86.64
87.92
DRAINAGE DISTRICT AT VERO
ST. LUCIE COUNTY, FLA.
Saw Grass Muck (shallow) underlain with well decayed Muck and Sand.
First Drained June 1914. Probable average depth of Drainage 2.5 feet.

FIG. 9.

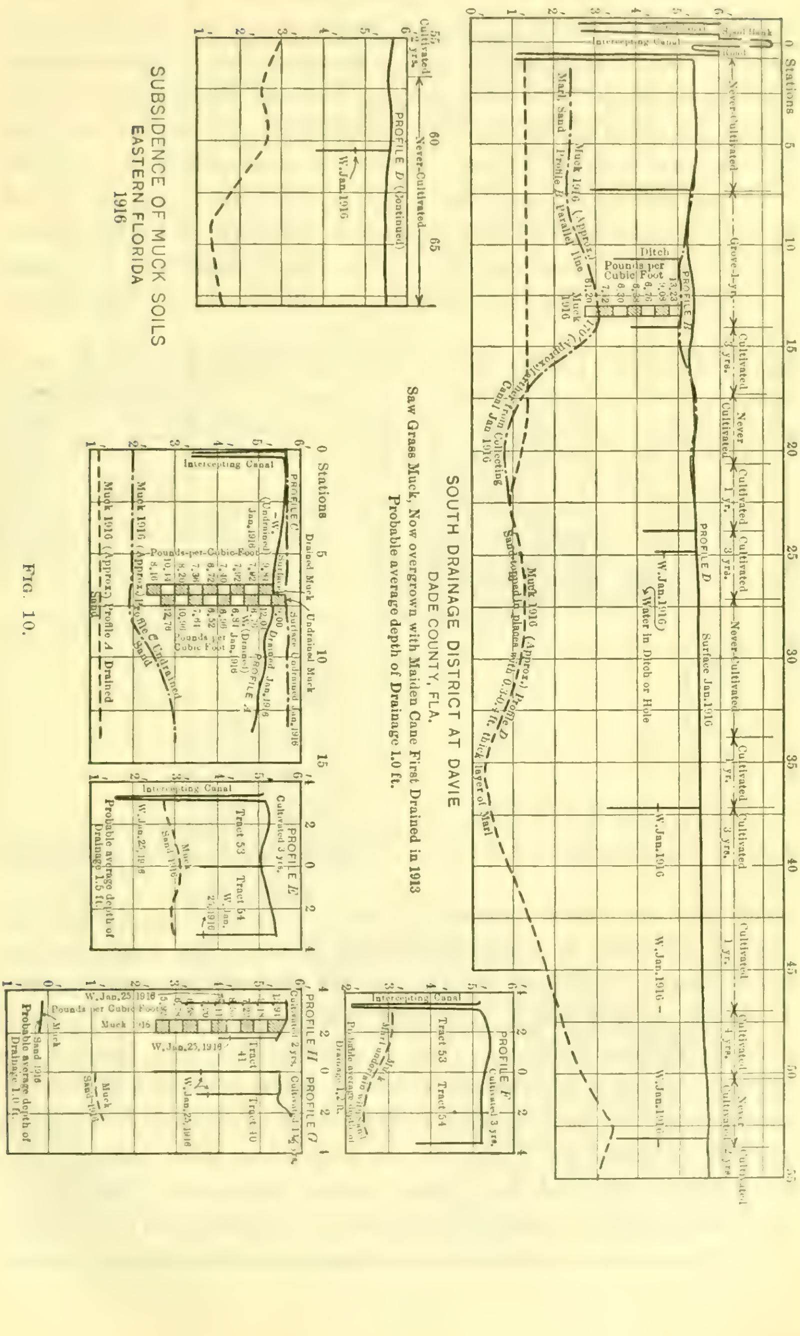

SUBSIDENCE OF MUCK SOILS
EASTERN FLORIDA
1916
SOUTH DRAINAGE DISTRICT AT DAVIE
DADE COUNTY, FLA.
Saw Grass Muck, Now overgrown with Maiden Cane First Drained in 1913
Probable average depth of Drainage 1.0 ft.
PROFILE B
PROFILE D
PROFILE D (Continued)
PROFILE C
PROFILE A
PROFILE E
PROFILE F
PROFILE H
PROFILE G
Intercepting Canal
Stations
Never-Cultivated
Cultivated
Grove-1-yr.
Surface Jan.1916
W.Jan.1916
Water in Ditch or Hole
Pounds per Cubic Foot
Ditch
Muck 1916 (Approx.) Profile D
Sand-topped in places with 0.30-4 ft. thick layer of Marl
Tract 53
Tract 54
Probable average depth of Drainage 1.5 ft.
Probable average depth of Drainage 1.0 ft.

FIG. 10.

The number over the title of each profile on Figs. 1 to 6 corresponds with the number on the map of Southern Louisiana, Plate XV, showing the reclamation districts. The title of the Florida districts, Figs. 14 to 18, gives the number and name of the district, and the county in which it is situated. The date on which the land was first drained is stated on each profile; in practically every case the land was saturated continuously before drainage. The datum for each profile is arbitrary, and is chosen so that zero is below any elevation platted. On several of the districts in Louisiana the levels were referenced to the Cairo datum. In running the Florida profiles,

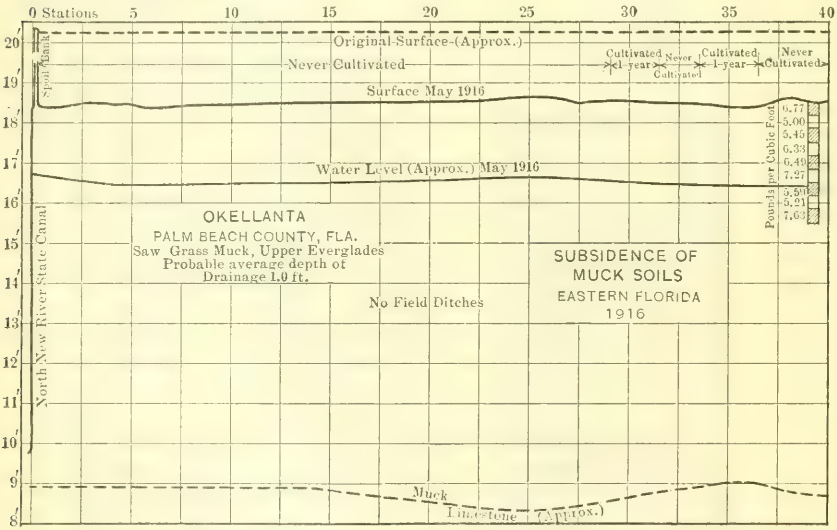


FIG. 11.

access was not always possible to exact mean sea level datum, but that, in all cases, is approximately the datum. The upper line on nearly every profile is the elevation of the original undrained surface. Where the district was only recently drained, the present surface elevations were usually so near the original that a separate line for the original surface is not shown. It will be noted that the line showing the elevation of the original surface is usually marked "Original Surface (Approx.)." This does not mean that there is any doubt about the elevation of the original surface, but merely that the line does not represent an actual profile taken in the field prior to drainage. In the Louisiana investigations the elevation of the original surface

RECLAMATION DISTRICTS

NO.	PARISH	NAME OF DISTRICT	ACRES	REMARKS
1	Calcasieu	Reclamation District No. 1. Lake Charles	700	
2	Cameron	Cameron Drainage District No. 2	14,730	
*3	Vermilion	Reclamation District No. 1 & 2, Florence	7,400	
*4	St. Mary	St. Mary Drainage District No. 3	19,000	
5	St. Mary	Avoca Drainage District	15,600	
6	Terrebonne	Upper Terrebonne Drainage District	4,240	
7	Lafourche	Reclamation District No. 7, Raceland	3,840	
8	Lafourche	Reclamation District No. 6, Raceland	4,760	
9	Lafourche	Reclamation District No. 4, Raceland	4,240	
*10	Lafourche	Reclamation District No. 3, Raceland	2,400	
11	Lafourche	Reclamation District No. 1, Raceland	800	940 acres in cultivation with 1,300 acres being added to district.
12	Lafourche	Reclamation District No. 2, Raceland	2,300	
13	Lafourche	Reclamation District No. 5, Raceland	1,320	
*14	Lafourche	Smithport Plantation	647	
15	Lafourche	Fay Plantation	2,000	300 acres in cultivation District to be enlarged
16	St. Charles	St. Charles Municipal Drainage Dist. No. 2	7,150	
17	St. Charles	St. Charles Municipal Drainage Dist. No. 1	9,860	Contains Sub-District of 800 acres in cultivation.
18	St. Charles	St. Charles Drainage District No. 1	2,840	
*19	Lafourche	Des Allemands Drainage District No. 1 & 2	3,320	1,880 acres under pump 1,440 acres being added
20	Lafourche	Des Allemands Drainage District No. 3	5,400	
21	Lafourche	Gheens Plantation	7,600	
22	Lafourche	Reclamation District No. 5, Larose	2,700	
23	Lafourche	Reclamation District No. 4, Larose	2,560	
24	Lafourche	Reclamation District No. 1, Larose	640	
25	Lafourche	Reclamation District No. 2, Larose	2,720	
26	Lafourche	Reclamation District No. 3, Larose	2,560	
27	Lafourche	Reclamation District No. 2, Golden Meadows	1,200	
28	Lafourche	Reclamation District No. 1, Golden Meadows	1,780	
29	St. Charles	La Branche Drainage District	7,500	
30	Jefferson	Kenner Drainage District	6,300	
*31	Jefferson	Willswood Plantation	2,000	
32	St. Charles	Lake Catsouache Reclamation District	5,000	
*33	Jefferson	Jefferson Drainage District No. 2	5,000	
34	Jefferson	Jefferson Drainage District No. 3	5,100	
*35	Orleans	New Orleans Land Co. District No. 1	1,380	
*36	Orleans	Little Woods District	7,600	
37	Plaquemines Jefferson	Plaquemines-Jefferson Drainage District	37,750	
*38	Plaquemines	Reclamation District No. 1, Poydras	2,500	
39	St. Bernard	Reclamation District No. 2, Poydras	15,000	
40	St. Bernard	Shell Beach Drainage District	7,950	
41	St. Bernard	Gillem Plantation	3,800	
42	St. Bernard	Bayou La Loutre Drainage District	7,000	
43	Plaquemines	Venice Drainage District	1,100	

*Districts in which records of rainfall and pumping are being kept by U.S. Dept. of Agriculture

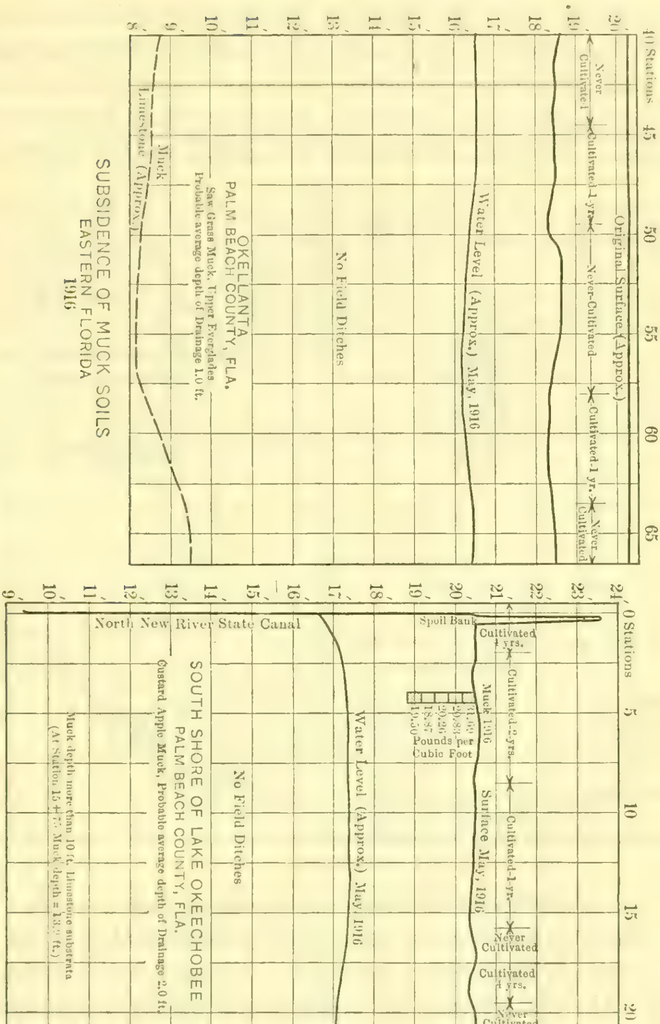

SUBSIDENCE OF MUCK SOILS
EASTERN FLORIDA
1916
OKELLANTA
PALM BEACH COUNTY, FLA.
Saw Grass Muck. Upper Everglades
Probable average depth of Drainage 1.0 ft.
40 Stations
45
50
55
60
65
Original Surface (Approx.)
Never Cultivated
Cultivated-1 yr.
Never-Cultivated
Cultivated-1 yr.
Never Cultivated
Water Level (Approx.) May, 1916
No Field Ditches
Muck
Limestone (Approx.)
SOUTH SHORE OF LAKE OKEECHOBEE
PALM BEACH COUNTY, FLA.
Custard Apple Muck, Probable average depth of Drainage 2.0 ft.
0 Stations
5
10
15
20
North New River State Canal
Spoil Bank
Cultivated 4 yrs.
Cultivated-2 yrs.
Cultivated-1 yr.
Never Cultivated
Cultivated 4 yrs.
Never Cultivated
Muck 1916
Pounds per Cubic Foot
Surface May, 1916
Water Level (Approx.) May, 1916
No Field Ditches
Lagoon

FIG. 12.

was usually determined by a profile taken on the undrained land parallel with the one on the drained land and as close as possible to it. Due to the uniformity in the elevations of the land, it is considered that the error introduced by this method would not be greater than 0.2 ft., except for small surface inequalities. Practically all the muck lands of Southern Louisiana are only a foot or two above sea level, and, as shown by the profiles, are almost flat. Before drainage the lands are saturated continuously and usually flooded several inches

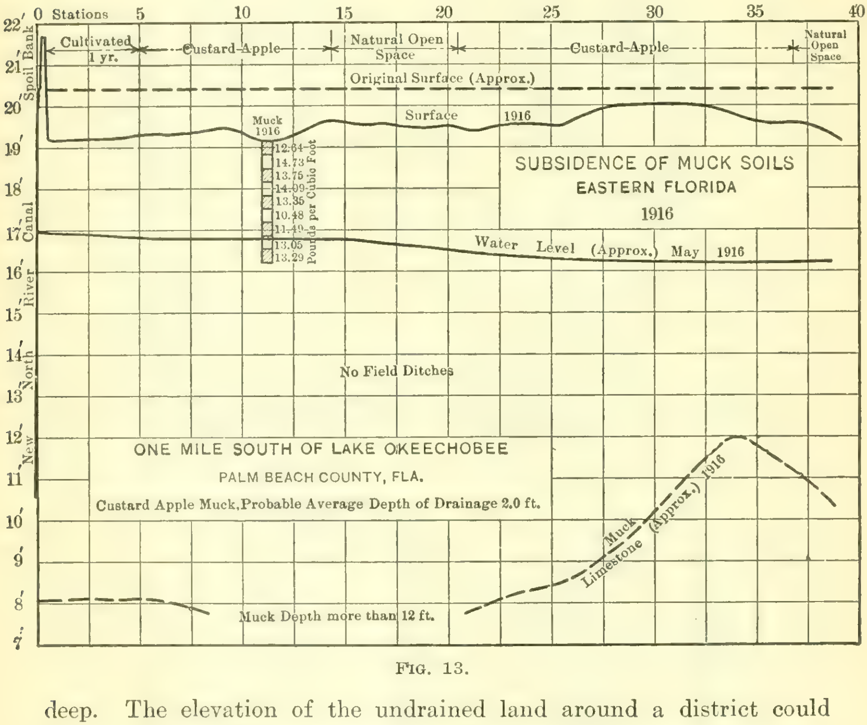


FIG. 13.

deep. The elevation of the undrained land around a district could change but very little, except for a slight subsidence due to the slight surface drainage furnished by the canals along the levees in the drained district. Any change that did occur would make the apparent subsidence of the drained land less than it really was. In running the profiles on both the drained and the undrained lands, the elevations on lines perpendicular to the profile were taken at frequent intervals, which showed that the land was level crosswise to the profiles. Where the profile is marked "Original Surface", data were available which

were taken before the land was drained. Although the elevation of the lands examined in Florida is considerable, such lands, also, are almost flat. In nearly every case the "Original Surface (Approx.)" is platted from profiles run on the center line of canals since constructed. As shown on Figs. 14 to 18, the profiles were then run parallel to these canals and a short distance from them. The same precautions as in

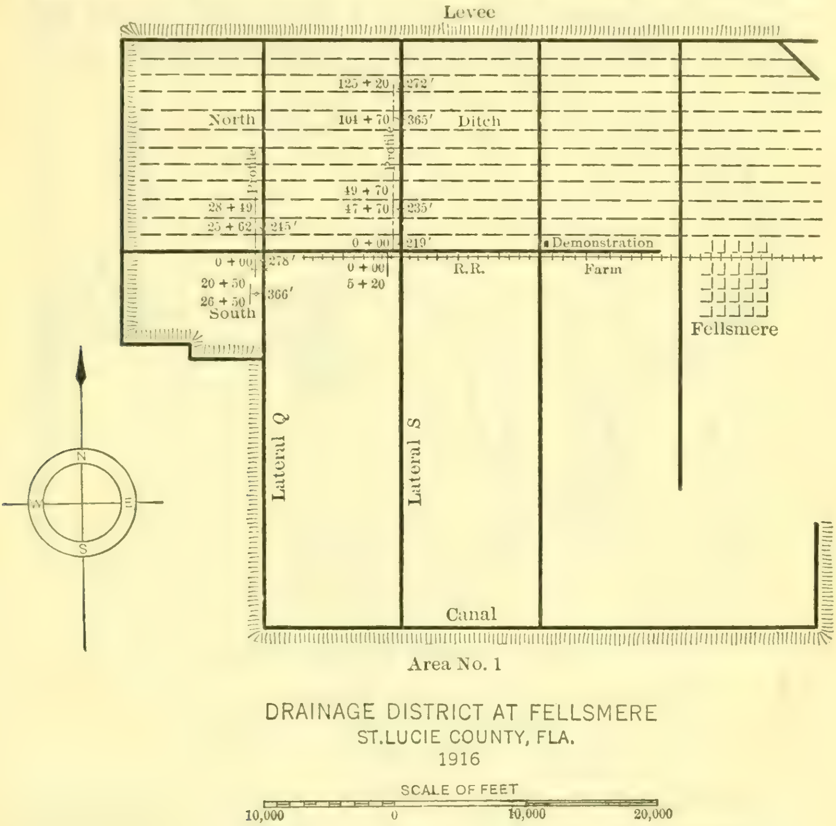


FIG. 14.

the work in Louisiana were taken in order to make sure that the land did not slope crosswise to the line of profile.

The second line from the top shows the surface elevation at the time the profiles were run. Notes indicate how long the surface had been cultivated at that time. The depth and spacing of the field ditches are shown on some of the profiles by vertical lines drawn to

scale; on others the depth and spacing are indicated by a note. The small maps, Figs. 14 to 18, show the direction of the small field ditches with respect to the profile. Usually, the surface near the deeper ditches and canals is lower, because that portion has been drained deeper and longer and has been cultivated for a longer time. This is especially noticeable on District No. 10 (Fig. 3). Here the large collecting ditches are at intervals of about half a mile. From the first they have been kept at a depth of about 3 ft. The small field ditches

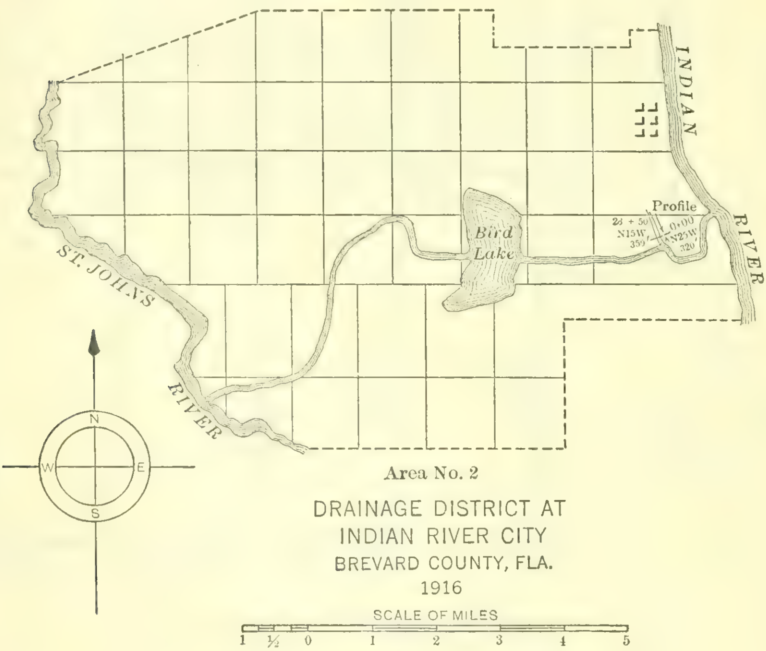


FIG. 15.

between these collecting ditches have been in very poor condition most of the time since the first drainage, the average depth of drainage being about 1 ft. until the latter part of 1914. However, when it was decided to cultivate the land back from the collecting ditches, the field ditches were cleaned out to a depth of nearly 3 ft. After a few years more of cultivation this difference in subsidence in this particular district will be less noticeable, and will eventually disappear.

The approximate location of the water-table is shown by another line. As the 2 months previous to the field examinations were with-

out rain, the water-table was uniformly lower than usual. Where this line is marked "Average depth of drainage" it refers to the average condition, rather than to the particular one which existed when the examinations were made. The standard depth for field ditches in these lands has been 3 ft. However, the ditches deteriorate very rapidly, and their average depth has been about 2 ft. In most of the districts the average depth of drainage has been not more than 2 ft.,

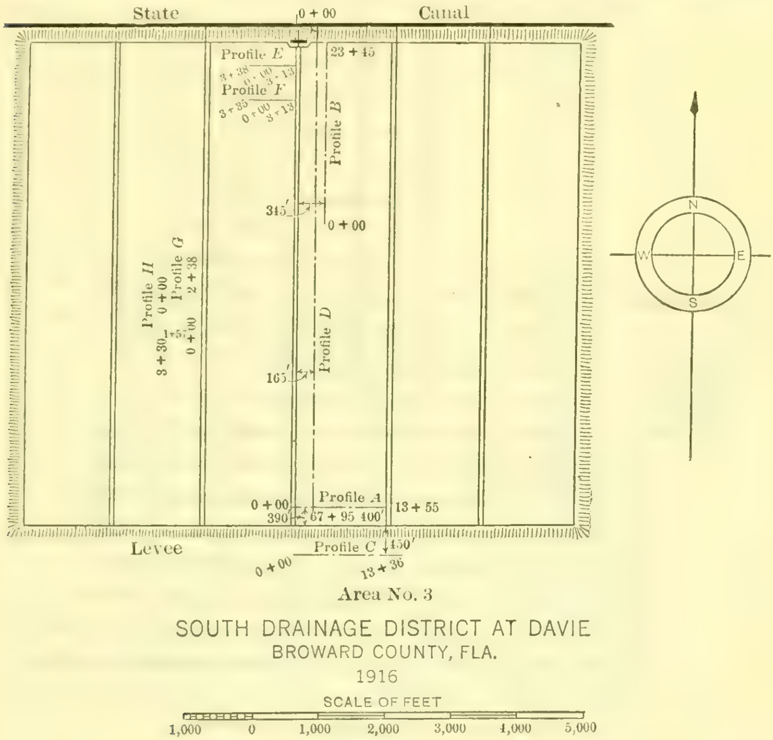


FIG. 16.

and for a large portion of the time it was only 1 ft. As the surface subsided the ditches had to be continually deepened, in order to afford even the shallow depth of drainage just mentioned. As a general proposition, it might be stated that the lands on which the profiles were run suffered from too shallow drainage during a portion of the time since they were first drained, and the water-table was probably never lowered farther below the surface than would be considered good drainage practice.

The approximate boundary between the muck and the underlying strata is shown by still another line. Borings and excavations were made at frequent intervals, but there was no attempt to get a com-

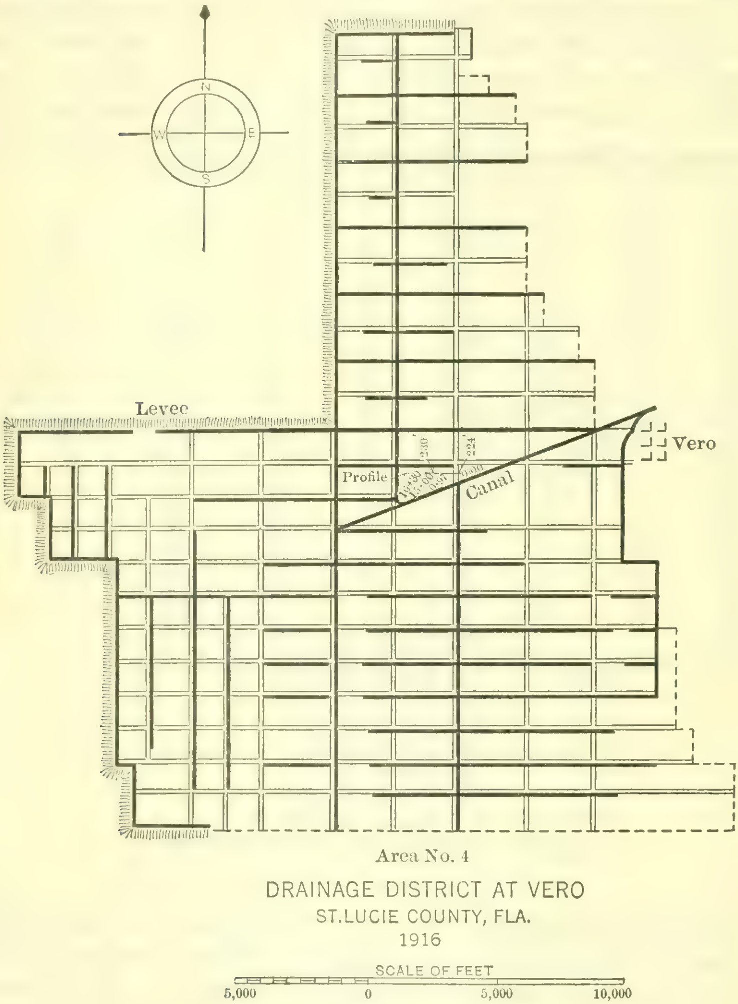


FIG. 17.

plete and accurate profile on the bottom of the muck. Quite often, the muck merges gradually into the silty clay or sand, so that the boundary between the two can be determined only approximately. The error in the location of this line is probably not more than 0.5 ft.

Where the districts had been cultivated so long that the muck had practically disappeared, this line is omitted from the profile.

Samples of soil were taken to as great a depth as possible, in both the drained and undrained muck, and in such a manner that they form a continuous column from the surface downward. Each sample had a depth and diameter of 4 in., except on the first district examined in Louisiana, where the depth was 6 in. In a few of the districts, a small layer, perhaps 1 in. thick, intervened between the samples, but, though allowance is made for this layer, in the depth of the sample below the surface, it is considered that this does not affect the accuracy of the determination of the average weight of the soil. The samples were all sun-dried as rapidly as possible, and when weighed were apparently completely dry. The weight of dry material per cubic foot of original sample was then calculated. All these samples are shown on the profiles at their true vertical scale and location, with the weight of dry material per cubic foot. By adding the weights of the various layers and correcting for the fact that the thickness of each is not a foot, the total weight of a section of a given depth can be obtained. Especial attention is called to the weight per cubic foot

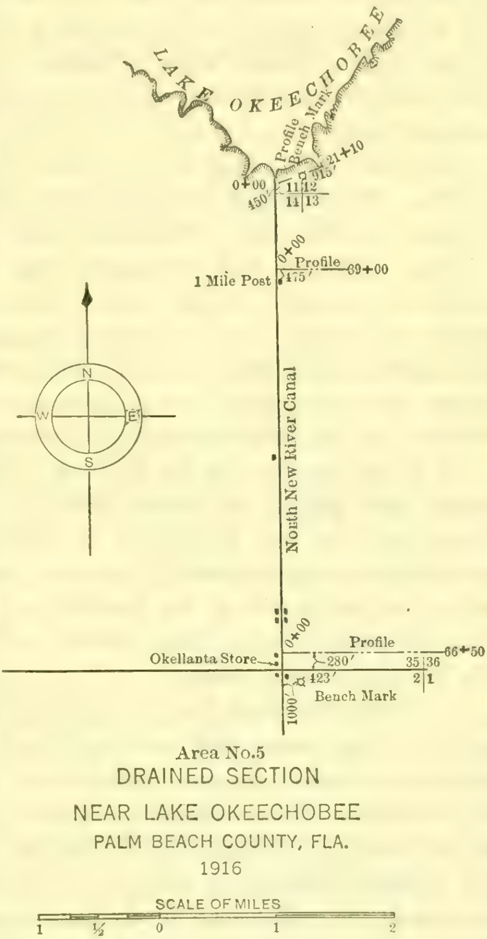


FIG. 18.

of the top layer of drained and cultivated muck. The soils which have been cultivated longest are the heaviest. The weight of a column of muck before and after drainage and cultivation can be obtained by taking the weights above a given level. This should be taken as deeply as the samples are given for the undrained muck, so as to get as low as possible in the drained portion. The layers of muck in the drained portion show an increase in weight a good distance below the surface. In most cases the column weighs less after drainage and cultivation, due to loss in weight by decay.

The small maps, Figs. 14 to 18, show the locations of the Florida profiles. These profiles were referenced so that they can always be relocated in the field. Permanent bench-marks were set near all of them. Thus it will be possible from time to time to measure additional subsidence of the surface shown on these profiles. Opportunity was offered in 1916 to repeat the profiles on the first district examined in Louisiana. The profile shows very clearly that a lowering of the water-table changed the elevation of the deep muck on this district. As yet, there has been no repetition of the other profiles.

It is the intention of the Division of Drainage Investigations to repeat these profiles at regular intervals for a long term of years. Where sufficient time has elapsed to make a considerable change in the elevation and character of the soil, samples of the soil will be taken, and the weight of dry material will be determined, as in the original investigations These profiles will thus form a history of the drainage of each district.

It is clearly evident that in planning drainage improvements for areas of deep muck land, some provision should be made for the gradual but certain decrease in elevation of the surface. In relatively small districts, where drainage is secured by pumps, this decrease can be met easily by lengthening the suction pipes on the pumps. As the drainage channels in such soft soils require considerable maintenance in the earlier years of drainage, they can be deepened accordingly. Where the land is drained by gravity, the elevation of the water at the outlet is usually fixed, and a change in elevation of the land to be drained will mean a revision of the hydraulic gradient in the main drainage channels, with the consequent change in width and depth of the channels.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

MULTIPLE-ARCH DAMS ON RUSH CREEK, CALIFORNIA

Discussion.*

By L. R. Jorgensen, M. Am. Soc. C. E.†

Mr. Jorgensen.

L. R. Jorgensen,‡ M. Am. Soc. C. E. (by letter).§—Throughout the discussion there has been a tendency to compare the cost of multiple-arch dams with that of gravity dams. As a general rule, it can be said that the multiple-arch dam is the cheaper, the more expensive the building material. For dams of moderate height, it is probably always cheaper, but not for heights of more than 130 ft. The only sure way of finding out is to estimate the cost of each type for any given location.

The system of struts has not been designed to help to prevent settlement or sliding of the buttresses; it cannot do that, and it is not necessary, for, in these particular cases, the rock foundation is possibly twice as strong in shear and crushing as the concrete in the buttresses. The struts are to stiffen the buttresses, and to transmit any unequal arch thrust which normally might exist at any place, due, for instance, to the adjacent arches being built, during periods of different temperatures, into the side-hill where the struts are anchored. The two struts close to the arch (Fig. 4) will also transmit to the side-hill the abnormal unbalanced thrust which might occur should one arch accidentally fail, thereby preventing a complete collapse of the dam. Should the struts fail at the same time as the corresponding arch—which is probable—the unbalanced thrust would

* Discussion of the paper by L. R. Jorgensen, M. Am. Soc. C. E., continued from August, 1917, *Proceedings*.

† Author's closure.

‡ San Francisco, Cal.

§ Received by the Secretary, September 4th, 1917.

Mr. Jorgensen. have to be taken up in tension by the steel in the line of struts on each side of the break. The eight 1-in. square rods should be able to hold 400 000 lb., as there are no laps between the knee-braces belonging to the different buttresses. The triangular girder mentioned on page 418* simply stiffens the up-stream ends of the buttresses so that they will stand up between points of application of the struts. The struts then transmit the unbalanced thrust to the side-hill, either by tension or compression, or by both.

Mr. Blackwell's criticism as to spillway capacity has been answered by Mr. Duryea. There occurs no precipitation of such quantity as to cause flood when the reservoir is full. During the season of maximum ice pressure—that is, in late winter or early spring—Gem Lake Reservoir will be nearly empty. Agnew Lake Reservoir is to be used as a forebay, and the daily fluctuations will break up the ice; therefore, it has not been deemed necessary to design these dams for ice pressure.

In comparing the three forms of multiple-arch dams mentioned by Mr. Flinn, the writer regards the one with the sloping up-stream face as the best and only one to consider. In this construction, the water pressure acting on the sloping face is the largest single factor contributing to stability, and this factor would be absent in a dam having a vertical face.

As to the leakage of different types of dams, this will depend much more on the care with which they are constructed than on their type and design.

The writer agrees with Mr. Scheidenhelm, that, after all, in the case of multiple-arch dams, one must, for perfect safety, insist on an unyielding foundation.

As an example of the influence of the time factor, let it be assumed that an arch is subjected suddenly, or during an interval of a few hours, to a temperature drop, penetrating through the arch material, of, say, 80°, sufficient to cause a maximum tension in the concrete of, say, 300 lb. per sq. in. If this same temperature drop of 80° was brought about during an interval of 20 or more days, the tension in the concrete would only amount to about 100 lb. per sq. in., and, naturally, the tendency to form cracks would be much less than in the first case. Daily temperature changes will hardly penetrate the arch sufficiently to have any practical effect, but seasonal temperature changes will, and here is where the action of the time factor tends to lessen the tendency to form cracks by making the concrete stretch more for the same stress, because the change has been slow. The publication by Mr. F. R. McMillan, referred to on page 410,*

* *Proceedings*, Am. Soc. C. E., March, 1917.

Mr. Jorgensen

gives detailed results of numerous tests of this nature, and is very interesting.

The multiple-arch dam at Ogden, Utah, referred to by Mr. Wegmann and Mr. Williams is of unusually conservative design, and it is no wonder that the bid price was only from 12 to 15% lower than the bid price on an ordinary gravity dam for the same site, as Mr. Goldmark's design surely has a very much larger factor of safety than the ordinary gravity dam.

Mr. Douglas says, "The unit price of masonry for the multiple-arch dam is out of all proportion to that of the gravity dam, as illustrated by the price of $22 per cu. yd. given by the author." This high price, however, is due to the high cost of the material at this particular place (Gem and Agnew Lakes), and not to the type. A multiple-arch dam in an accessible location, and where conditions are ordinarily favorable, can be constructed for $13 per cu. yd., and this price may possibly include the steel reinforcement.

Mr. Douglas is correct in stating that frictional resistance can only act after the section has failed under shear. One should rely on shear or friction. The maximum unit shear of approximately 106 lb. per sq. in. existing at Elevation 80 below the crest can hardly be considered excessive. In this case the area of the arch ring has been added to the area of the buttress proper, as both must act to resist shear at the same time. The fact that there is a very large vertical load should be of some comfort, as it improves the ability of the buttresses to withstand actual shear.

The stress on the buttresses is given between two limits on pages 416 and 417.* The following formula (which can be found in some Handbooks) gives the maximum stress on any horizontal plane more definitely:

$$\text{Ideal maximum stress} = \frac{m-1}{2m} \times \text{unit vertical compression}$$

$$+ \frac{m+1}{2m} \sqrt{(\text{unit vertical compression})^2 + 4\,(\text{unit horizontal shear})^2}$$

(m = Poisson's ratio = 5 for concrete). Then, at Elevation 80,

$$\text{Ideal maximum stress} = 0.4 \times 192 + 0.6 \sqrt{192^2 + 4 \times 106^2}$$
$$= 76.8 + 171.6 = 248.4 \text{ lb. per sq. in.}$$

It is true, as pointed out by Mr. Nishkian, that the stress produced by the normal component of the arch thrust on the buttress is greater than this ideal stress. This, however, is the case only toward lower elevations, where the "column" is very short; the normal component of the arch thrust diminishes rapidly toward higher elevations. The

* *Proceedings*, Am. Soc. C. E., March, 1917.

Mr. Jorgensen.

ideal maximum stress of 248.4 lb. per sq. in. diminishes less toward the upper system of struts; and, although the column ratio is about 20, there might be, and very likely is, some column action due to the ideal maximum stress.

Both Mr. Nishkian and Mr. Williams point to the elliptical arch as being the most correct form. Thus far, the writer has contended that the additional work of making a three-centered arch would outweigh any possible gains; however, he will soon see how the elliptical arch works out on a multiple-arch dam structure with the construction of which he is connected.

The writer cannot agree with Mr. Williams that the arch can be of uniform, or diminishing, thickness downward from the point of one-third the height of dam. Even though the stresses may not be excessive, the penetrating power of water increases with the depth, and, therefore, the thickness of the wall should increase logically. It must be kept in mind that even a sweat on the down-stream face will produce icicles in cold weather, and it is not yet known to what extent, if any, this will damage good concrete. It is known that it damages poor concrete. Good concrete is fairly water-tight, and gunnite is remarkably so, but, nevertheless, it requires great care to construct a water-tight structure. Some of the arches on the Gem Lake Dam are absolutely tight, some of them sweat in places, and a few drip in places. A few small springs have formed behind the dam, and a trickle of water comes under one arch; but, all told, the total leakage is very small.

Some tests were carried out in the Laboratory of the University of California on the water-tightness of plaster "shot" on the dam face with a cement gun. Several plaster slabs, from $\frac{5}{8}$ to $1\frac{1}{2}$ in. thick, made at Gem Lake, were tested with water-pressures ranging from 700 to 1 600 ft., for several hours, with no moisture coming through the slab. One 1-in. slab held a head of 1 610 ft. for $2\frac{1}{2}$ hours without showing moisture, then the water pressure was raised gradually to 3 400 ft., and the specimen broke in bending, having leaked a little just before breaking.

Mr. Howson states the case correctly when he says that a rock-fill dam is best adapted to a site where the underlying bed-rock across the floor of the canyon is covered by, say, from 20 to 30 ft. of over-burden, as it is only necessary to carry a thin cut-off wall down through the overlying material.

The author thanks those participating in the discussion for the interest shown.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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PAPERS AND DISCUSSIONS

AN AERIAL TRAMWAY FOR THE SALINE VALLEY SALT COMPANY, INYO COUNTY, CALIFORNIA

Discussion.*

By F. C. Carstarphen, M. Am. Soc. C. E.†

F. C. Carstarphen,‡ M. Am. Soc. C. E. (by letter).§—The writer appreciates the favorable tenor of the discussion which has been presented. Several points mentioned by Mr. Lamb are worthy of note. The first deals with the question of the price of salt, which no doubt was of great importance to the promoters of the Saline Valley Salt Company, but was only of passing interest to the builders of the tramway. Information as to the prices which can be obtained for salt on the Pacific Coast shows that they range from $3.50 to $40.00 per ton, depending on the grade and character. Ocean salt sells for about $3.50 per ton, which is somewhat less than the computed retail price of $26.70 per ton for salt, at the rate of 7½ lb. for 10 cents. It is the writer's understanding that milled salt suitable for table use sells wholesale at from $16 to $20 per ton, so that with the ample demand of the fisheries and other users of salt on the Pacific Coast and in the Northwest territories, the development of the Saline Valley salt field was considered by its promoters to be a worthy business enterprise. Mr. Carstarphen.

Mention has been made of the length of the tramway built for the American Copper Company, of Grand Encampment, Wyo. The writer quotes from a report on file relative to this line:

"The tram was built in four sections of about four miles to each section. The first section from the smelter to Station 1 was approximately four miles in length, with a raise of perhaps 4% across an almost level prairie. The buckets were transferred there from Section

* Discussion of the paper by F. C. Carstarphen, M. Am. Soc. C. E., continued from August, 1917, *Proceedings*.

† Author's closure.

‡ Trenton, N. J.

§ Received by the Secretary, September 5th, 1917.

Mr. Carstarphen. 1 to Section 2, etc. Section 2 was almost four miles, and the first three miles above Station 1 was a gradual raise of probably 4 per cent. For the last mile of Section 2 the raise was probably 12%, and crossed one or two deep gulches. Station 2 to Station 3 was over a rough broken country. It consisted of long spans and some very high towers. From Station 3 to the loading station the tramway crossed the Continental Divide. Station 3 was in a canyon, and the mine was also in a canyon. The tramway line raises probably 3 000 ft. between the two terminals along Section 3.

"This tramway is of the old Leschen lug type; one of the first four cable tramways built by that company. The lugs consist of a piece of sheet steel bent around the cable and a cast-iron button or lug riveted into this steel, and this button in turn entered into channel irons on the buckets by means of triggers and slide bars. There is one of the same type of tramways at the Sunnyside Mine, Silverton, Colo.

"The cables on the tramway at Encampment were of the 6 by 7 long lay type, and probably $1\frac{1}{8}$ or $1\frac{1}{4}$ in. in diameter. The traction ropes were $\frac{3}{4}$ in. in diameter.

"Tram was driven by power generated from boilers located at the various stations along the line.

"Mr. Riblet was on the ground, and designed the structures and figured all the towers, angles, and deflections of the cables. Equipment was all furnished for this line by the A. Leschen and Sons Rope Company, of St. Louis, Mo.

"The traction rope on these towers was supported by small sheaves which hung on a cross-beam 21 in. from the top of the stationary cable. These sheaves were grooved, and when the buckets passed over the sheave the lug would raise the rope high enough to pass between the rope and the sheave. One main difficulty was this: These channel irons on the buckets were bolted to the frames with $\frac{1}{4}$-in. carriage bolts, two on each side, or four bolts altogether. The track ropes were so slack that the buckets in passing over the long spans would hang down so low that the traction rope would pull the lugs through these irons, breaking off the bolts and letting the buckets run away. The lug, being riveted to the cable, would immediately whirl over a few times and, as it passed, the next tower would hang on a rod that supported the timber for the tower sheave. Towers were all set on the ground without any anchorages, and consequently it dragged this tower to the next one and then hooked onto that one and pulled it to another one. Sometimes the writer would go up along the line and find as many as four towers all piled up in one bunch. The engines were powerful, and the man in charge had no idea when anything was hung up. They just kept pulling as long as they could move the thing.

"The downward pressure on the traction rope in coming over the Continental Divide, caused by the driving station and the loading station being in the canyon below, and long spans leading to each one of them, was so great that it would simply cut through a new tower sheave in a very few hours. We experienced most of our difficulties in the operation of this tramway on the three miles of country adjacent to the Continental Divide."

The topography, due to the erosion of the Bridger sandstones, is not comparable with the faulting and mountain-forming tendencies which have developed the extreme slopes and elevations characterizing the mountainous areas of California; and, therefore, the difficulties of design of the two tramways are not of the same order. Mr. Carstarphen.

No difficulties were experienced on the Saline Valley tramway due to the faltering of the carriers in passing the saddles supporting the track cables, on account of the traction rope not being parallel with the bearing cable; nor is this tendency of any importance on tramways of the Trenton-Bleichert construction, for it is eliminated by the fundamental principles of tramway design. Reference to Fig. 6, which represents loaded and empty carriers passing an 80% grade, will show that the track cables and traction rope are practically parallel.

In modern tramway design, it is customary to place the cable supports so that the angle formed by the intersecting tangents developed by the chord slope and the deflection of the empty and loaded cable, is kept within a reasonable amount, depending on the radius of the saddle and its capacity in angular magnitude.

The tensions used in tramway track cables have changed from time to time. Twenty years ago track cables were erected with a tension as low as 14 500 lb. per sq. in. Tensions were then increased to 22 500, 25 000, and 30 000 lb. per sq. in. Beyond this stage increased knowledge was obtained of the stresses developed in locked coil cables due to bending. The influence of frictional hysteresis on the modulus of elasticity of the cable in bending was ascertained, so that the question of the proper tension of a tramway track cable depends largely on the condition of the particular tramway which is being designed.

If it is admitted that stress, due to bending, is developed in the outermost wire of a track cable, it is consistent to assume that a cable can be made to fail due to excessive bending only. It is well known that, if sufficient direct stress is developed in the cable, it can be pulled in two. Therefore, there is an economic tension to be applied to the track cable which will develop a minimum combined stress in the outermost wire due to direct load and bending.

Bending stress is developed in the track cable due to its support of the loaded carrier. The measure of this bending is a cable curve. The tangent of the intersection angle is equal to the weight of the load divided by the horizontal component of the tension in the track cable. If the loaded carriers are low in weight and the tension in the track cable is high, the intersection angle of the curve under the carrier will be small; that is, the curve will have a long radius, and the stress developed in the outermost wire, due to bending, will be less than that due to direct tension.

Mr. Carstarphen.

This is the fundamental criterion which should be considered in designing aerial tramways, where it is desirable that a proper life of track cable should be obtained, together with a minimum of interference due to improper elevation of tower saddles.

The spacing of the towers depends on the tension in the track cables and the weight of the load supported. With a proper radius for the saddle supporting the track cables, the spacing of the towers depends on the economic construction height of these supports. The farther apart the towers are, the greater must be their height in order to balance the deflection and clearance required between the bottom of the carrier and the ground surface. The longest span which the American Steel and Wire Company has put in is 3 600 ft.

Tension stations may or may not be used, at the designer's option. In fact, it is customary to control the change of position assumed by the track cable when loaded and empty by erecting it as an anchored span. It is well known that the product of deflection and tension at any point on a suspended cable is constant. Accordingly, in anchored spans, if the loading changes, the deflection tends to remain a constant, due to the change in tension. If the cable were non-elastic and symmetrically loaded, there would be no change in deflection between the empty and loaded positions. In practice, there is a slight change in these positions, due to the elastic properties of the cable.

The pronounced limitation of tramway location to a straight line is no longer apropos, as present-day designers do not hesitate to handle moderate angles on the tramway supports. In this manner angles of considerable magnitude may be introduced into the alignment of the tramway without interfering seriously with its operation; nor is it necessary in cases of this sort to build a special structure called an "angle station."

Although the discussion has not been extensive, it has served a useful purpose in demonstrating that the proper design of aerial tramways is based on sound principles of engineering, and that such systems of transportation are not to be considered as expedients to be adopted when no other method can be used. Aerial tramways are safe, sane, and satisfactory.

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PAPERS AND DISCUSSIONS

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MODERN PRACTICE IN WOOD STAVE PIPE DESIGN AND SUGGESTIONS FOR STANDARD SPECIFICATIONS

Discussion.*

By Messrs. O. P. M. Goss and W. H. R. Nimmo.

O. P. M. Goss,† Assoc. M. Am. Soc. C. E. (by letter.)‡—The writer has read this paper with considerable interest. It contains some very interesting suggestions, and is believed to be a step in the right direction, at least in so far as it suggests the standardizing of specifications covering the design and construction of wood stave pipe. This subject, however, is very important, and should receive thorough consideration before any definite specification is approved. Mr. Goss.

There are, for example, certain statements in the paper which are not based on the natural laws underlying the most approved use of wood. In offering the following comments for consideration, the writer has endeavored to set forth certain facts which cannot be ignored in the general discussion of wood stave pipe.

On page 565§ the author states:

"Fir and pine are pitchy woods, and it is impossible to obtain commercial run lumber without sap, pitch, pitch seams, pitch pockets, and knots. Under conditions of partial saturation, this lumber will not last, and, even with saturation, the pitch and sap will be the cause of deterioration. Most failures are attributable to this fact. There are conditions under which fir or pine will have a long life and give perfect satisfaction."

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* Discussion of the paper by J. F. Partridge, Jun. Am. Soc. C. E., continued from August, 1917, *Proceedings*.

† Seattle, Wash.

‡ Received by the Secretary, July 30th, 1917.

§ *Proceedings*, Am. Soc. C. E., April, 1917.

Mr. Goss.

Pitch does not in any way cause the deterioration of timber. Tests of recent date, made at the U. S. Forest Service Laboratory, indicate that wood containing resin, deteriorates a little less rapidly, on the average, than that which is free from this substance. The following quotation is taken from a recent report issued from the U. S. Forest Products Laboratory, at Madison, Wis.:

"*Relation of Resin to Strength and Durability.*—Data on the effect of resin on durability were worked up for 105 samples of long leaf pine. The results, when considered as averages for four durability classes, indicate that increasing amounts of resin tend to be directly correlated with increased durability. Individual blocks do not necessarily bear out this relation, showing that there are other factors involved."

The following quotation is from page 567* of the paper:

"The cases of redwood pipe already cited illustrate its adaptability, whether laid on the surface of the ground, partly or completely buried, or run through salt marshes or tropical swamps in direct contact with the soil humus. Direct exposure to the rays of the desert sun, and alternate wetting and drying when the pipe is used intermittently in irrigation systems, do not lessen its efficiency."

The writer cannot see how any one can make such a broad statement, and particularly that "alternate wetting and drying when the pipe is used intermittently in irrigation systems" does not lessen its efficiency. Again, tests made recently at the U. S. Forest Products Laboratory indicate that all woods are subject to decay under adverse conditions. It could only be considered good engineering when conditions are prevented which would tend to make any wood less durable. Heartwood from the California big trees, at the end of a 12-month test, showed a loss in weight of 35.1%, due to deterioration.† This wood is not the same as redwood, but is similar, and is used here in the absence of similar data for redwood. Western red cedar, an unusually durable wood, under similar conditions, showed a loss of 21.3%, but it is reported that the samples were too wet to give a fair test, which indicates that 21.3% loss is lower than a fair test would have shown. Port Orford cedar, readily conceded to be one of the most durable woods, showed a loss of 22.6%, which, again, is lower than the value would have been had the specimens been less saturated. Douglas fir showed a loss of 28.1%, according to these tests. Deterioration, in this case, also, was somewhat retarded by an excess of moisture. These results show that the most durable woods are likely to decay if subjected to adverse conditions. Due to this fact, no wood pipe, regardless of the species from which the staves are made, should

* *Proceedings,* Am. Soc. C. E., April, 1917.

† Table 7, p. 90, "Laboratory Tests on the Durability of American Woods," by C. J. Humphrey, of the U. S. Forest Products Laboratory.

be laid under unfavorable conditions without taking practical precautions against decay of the wood fiber. Mr. Goss.

On page 570* the author states:

"Fir and pine, being hard woods compared with redwood, and being coarse-grained, having wide rings of hard and soft wood, enter the classification of woods giving excessive percolation, with slow and incomplete penetration. This is caused by the water passing rapidly through the soft summer wood, appearing in drops on the outer surface of the pipe, and of penetrating but slowly, and often through only a fraction of a stave, along the hard winter rings. The result is a stave showing percolation and incomplete penetration at alternate points throughout its cross-section."

Douglas fir, as a matter of fact, is one of the most difficult woods to penetrate with a liquid, and, in this respect, might about as well be classed with metal as with pine. In creosoting timber, throughout the United States, there has seldom if ever been found a wood which has required so much scientific study to secure thoroughly satisfactory impregnation as has been the case with Douglas fir. In the treatment of ties of this wood, it is highly desirable to perforate the sides of each tie with fine holes, uniformly spaced, in order to get an effective injection of creosote oil.

It is usually specified that pipe staves of Douglas fir shall be practically free from all defects, which means that this stock must not be cut from the center of the log, which usually contains most of the knots and other defects. Due to this fact, the staves are cut from the fine-grained material found on the outer portion of the large fir logs, and not from the coarser-grained material, which is almost always confined to the center portions of the tree.

In the selection of pipe staves, care is taken to eliminate coarse-grained material. No difficulty whatever is experienced in eliminating practically all the sap wood, and, in pipe properly manufactured, the sap is never allowed to occur on the outer portion of the stave. Sap is not considered a defect on the inner portion, in a line which is in continuous service, because of the fact that, under this condition, it is always thoroughly saturated.

In Douglas fir staves of medium and fine growth, the summer and spring wood bands of the annual ring are so close together that if either is thoroughly saturated the adjacent one must also be wet.

The soft portion of the annual ring of redwood is more porous than the corresponding part of Douglas fir, as shown by Figs. 6 and 7. Redwood holds a natural moisture content of about 80% and the normal moisture content of Douglas fir is 33%, based on the dry weight of the wood in both cases. These facts indicate a greater porosity in redwood than in Douglas fir. As a matter of practice,

* *Proceedings*, Am. Soc. C. E., April, 1917.

Mr. Goss. however, neither of these woods is justly subject to criticism from the standpoint of excessive percolation.

The author refers to the "soft summer wood and hard winter rings." Technically, the summer wood is the hard part of the annual ring, and is formed during the dry period of the tree's growth; that is, through the late summer and early fall. The soft or porous portion of the annual ring is the spring wood, and is formed in the early spring and summer, when moisture in the soil is abundant and the growing rate of the tree is most rapid.

On page 571* the author states:

> "The lumber, therefore, should be perfectly dry before being used. It should be dried by the natural or air-drying process, not by the forced or kiln-drying process. By air-drying only, is perfect, sound, strong lumber obtained. Kiln-drying makes brittle and lifeless lumber. Air-drying requires time, and, as lumber should be seasoned for at least a year for the best construction, a large stock of it should be available at all times."

As a matter of fact, better results may be secured by correct methods of kiln-drying Douglas fir lumber than by air-seasoning it. Like any other process of manufacture, kiln-drying has its successes and failures. With a fundamental knowledge of wood and of the law governing successful kiln-drying, entirely satisfactory results are now obtained. It is possible to kiln-dry Douglas fir staves in such a way as to leave them in perfect condition for use. This kiln-drying may be, and is, done to-day so as to produce faultless lumber with its full original strength, in fact, much more than its original green strength, as it comes from the kiln.

Small specimens of long-leaf pine,† $\frac{3}{4}$ by $\frac{3}{4}$ in. in cross-section, air-dried for 98 days and re-soaked in water for 47 days, showed a crushing strength of 2 213 lb. The same material (matched pieces) after being kiln-dried 35 days and re-soaked in water 63 days exhibited 2 268 lb. Air-dried, re-soaked specimens of red spruce, handled in exactly the same way, showed a strength of 1 553 lb., and for the kiln-dried, re-soaked, 1 606 lb. Chestnut was also used in this test, and exhibited 1 482 lb. for the air-dried, re-soaked, and 1 573 lb. for the exactly the same way, showed a strength of 1 553 lb., and for the kiln-drying does not injure the strength of the wood, as compared to air-seasoning; and it must be remembered that these tests were made on specimens which were water-soaked after being kiln-dried and air-dried, which makes the tests particularly applicable to pipe staves in service.

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* *Proceedings*, Am. Soc. C. E., April, 1917.

† *Circular No. 108*, Forest Service, U. S. Dept. of Agriculture, Table 5.

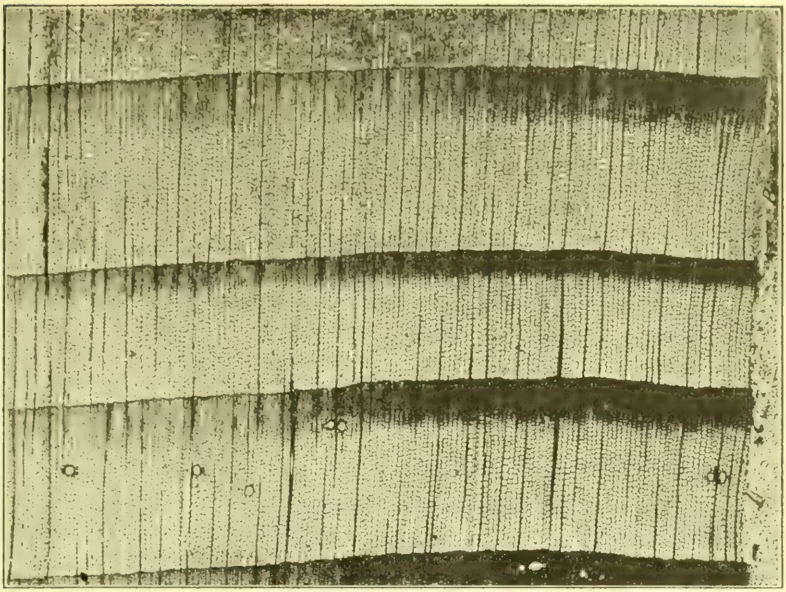


FIG. 6.—CROSS-SECTION, DOUGLAS FIR, SHOWING CELL STRUCTURE.
(20 DIAMETERS.)
A.—SUMMER WOOD. *B.*—SPRING WOOD.

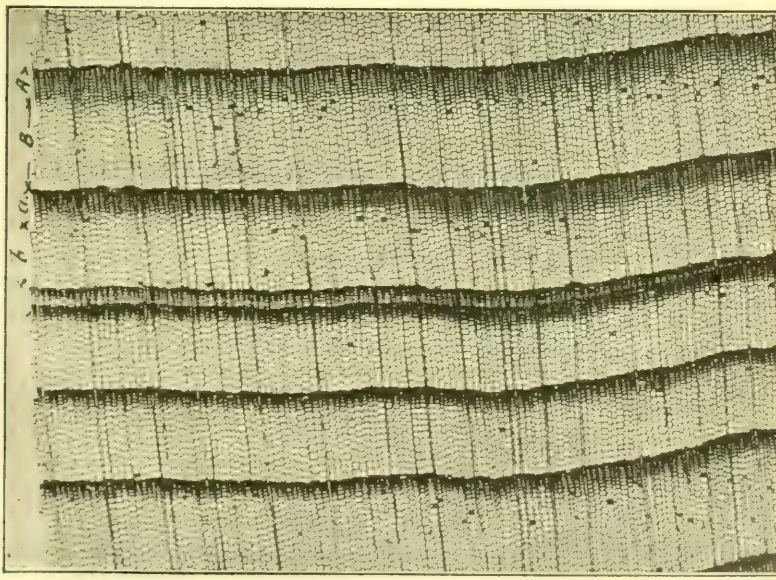


FIG. 7.—CROSS-SECTION, REDWOOD, SHOWING CELL STRUCTURE. (20 DIAMETERS.)
A.—SUMMER WOOD. *B.*—SPRING WOOD.

Mr. H. D. Tiemann, of the U. S. Forest Products Laboratory, one of the best versed men in the United States on the theory of kiln-drying lumber, states:* Mr. Goss.

"While air-drying is undoubtedly the safest method, the process is ordinarily so slow, requiring a year or longer, according to species and size, that forced 'artificial' drying becomes a business necessity. Moreover, air-drying is by no means always to be preferred to kiln-drying from the standpoint of the quality of the product."

The writer has made tests on Douglas fir and western hemlock, similar to those quoted from *Circular No. 108* of the U. S. Forest Service, showing that kiln-drying operations do not affect adversely the strength of either of these species, as compared to air-seasoned material. This may not be true in the case of redwood or cedar, as the structure of these woods is very different, and the cells appear to collapse under the application of heat very much more readily than with Douglas fir or pine.

On page 572† the author states: "For correct pipe design, only air-dried lumber should be specified." This seems to be so far out of line with good practice that it needs little comment. Practically all the Douglas fir lumber which is put into pipe is thoroughly kiln-dried before use, and any one familiar with this material knows that, under conditions which are favorable to any wood pipe, it gives the best of service.

The author discusses machine-banded pipe, and refers to the difficulty experienced with the banded couplings, due to their lack of durability. He suggests that fir pipe couplings be discarded and replaced by couplings of redwood. This is not the best solution of this problem. A better method is to creosote thoroughly the staves of pine, fir, or redwood before making up these collars. This will give them durability even greater than that of the pipe line itself under complete saturation. This is strictly an economical and efficient method, and is to-day being practiced by at least the most progressive pipe manufacturers on the Pacific Coast.

The following is quoted from page 577:†

"The use of the inserted or slip-joint pipe is not to be recommended. Such a connection weakens the end of every section, because nearly one-half of the shell of the pipe is cut away to make the joint. A reinforcing rod is often used to draw the joint tight, but if the male and female tenons are eccentric, leakage cannot be avoided."

The writer has used a considerable quantity of machine-banded Douglas fir pipe with the inserted couplings, and has found it to give thorough satisfaction. As to the strength of these joints, the writer

* *Bulletin No. 509*, U. S. Dept. of Agriculture, p. 2.

† *Proceedings*, Am. Soc. C. E., April, 1917.

Mr. Goss. has had no experience which would indicate that they are not sufficiently reinforced. Redwood staves are usually thinner than those of fir or pine, and this, together with the fact that redwood is weaker than Douglas fir or pine, might account for the lack of satisfaction given by this type of pipe. It is an entire success when made of Douglas fir, in accordance with present standards.

The writer cannot see any reason for leakage due to the tenons or cups being slightly eccentric with regard to the axis of the pipe, so long as they are circular and well manufactured. In his experience, no difficulty from leakage has been found.

Douglas fir has considerable advantage over redwood in strength in compression across the grain. The average strength of these woods* is 570 lb. per sq. in. for Douglas fir and 525 lb. for redwood. These figures show redwood to be approximately 92% as strong in side bearing as Douglas fir.

On page 583† the author states: "For permanent work, redwood should be selected, or fir or pine of high-grade staves kept saturated and well painted."

It would be inviting trouble not to apply the clause "kept saturated and well painted" to redwood, as well as to fir and pine. It is the weak points in a wood pipe that cause the trouble, and in good engineering these weak points should be eliminated by proper means. Wood staves used under low heads or under intermittent service should be creosoted by the pressure process. If, for any reason, this is impossible, a brush treatment with hot creosote, or carbolineum, should be applied to the edges and ends of the staves, and also to the entire outer portion of the pipe line. This treatment should be followed with a hot asphalt or tar coating thoroughly applied.

On page 572,† the following is found:

"In regard to the protection of the staves by applications of coatings of paint or disinfectant, little of value can be cited. Many claims are made for the benefits derived from various coatings, but sufficient data are yet lacking for reliable conclusions. It is certain that such protection increases the life of fir, pine, or other woods containing sap and pitch, but its merits on a redwood pipe have not yet been proved.

* * * * * * *

"A coating of at least $\frac{1}{16}$ in. should be the result of the first painting, and repeated examination should be made of the line, and the pipe painted every year or so."

It is safe to say that a protective coating or preservative properly applied will increase the life of any wood pipe. Mr. Partridge states

* *Bulletin No. 108,* Forest Service, U. S. Dept. Agriculture, Table 12.

† *Proceedings,* Am. Soc. C. E., April, 1917.

Mr. Goss.

that in fir and pine pipe it is necessary to apply a preservative coating "every year or so." The writer knows of no Douglas fir or pine pipe lines which have been coated as often as this. In fact, it would be impracticable and entirely unnecessary to paint a wood pipe line "every year or so" when such line is buried in the soil. If the proper material is used, the protective coating should remain on the pipe at least 15 years under such conditions. The writer has seen Douglas fir stave pipe uncovered 24 years after it had been laid, and the asphaltic coating was still in good condition. If the pipe is used above ground, it would not be necessary to paint it oftener than once in 5 years, because the natural conditions would tend to retard decay.

Untreated wood stave pipe has demonstrated its value when used with a thorough knowledge of the underlying principle controlling the efficient use of wood. There is in store for such pipe, however, a development which is now making its appearance and will materially change present practice, and that is the creosoting of the staves under pressure as a means of eliminating decay.

Considerable effort has been made by the Engineering Department of the West Coast Lumbermen's Association, particularly during the last 3 years, to improve the methods of creosoting Douglas fir in its various forms. One of the forms already studied is staves. In the methods of treatment which have been applied in the past (the old boiling or steaming processes), previous to the recent developments in the art of creosoting Douglas fir, there has been considerable loss in strength in compression perpendicular to the grain, due to the creosote treatment. This loss, of course, was not desirable in the case of staves, which depend on the strength of the wood in side bearing to resist the water pressure. There was a loss in strength of 30% or more in the staves as a result of either of these treatments. However, by the process now in use—the "boiling under a vacuum method"—which greatly reduces the temperature of the oil during treatment, or simply treating under low temperatures, there has been no loss in the strength of the staves.

The West Coast Lumbermen's Association recently completed some strength tests on staves which had been creosoted by this mild-temperature method of treatment. Twelve Douglas fir staves were selected, which were entirely free from sap wood, and twelve other staves were chosen as nearly like the first group as possible, except that they contained various quantities of sap wood. All these staves were 6 ft. long, and were kiln-dried. A section 1 ft. long was cut off the end of each stave and retained for a control test. The remaining portions of each of the twenty-four staves were treated in the following manner:

Mr. Goss.

They were warmed in creosote oil for 4 hours at 170° Fahr., and pressed from 0 to approximately 100 lb. per sq. in., until they had received about 16 lb. of oil per cu. ft.

The oil was then heated from 170 to 230° Fahr. in 3 hours, and held at this latter temperature for 1 hour. The staves were then removed.

The final heating bath was for the purpose of removing the surplus oil and cleaning the stave.

TABLE 4.—Band Bearing Test on Douglas Fir Staves, Natural and Creosoted. Staves Soaked in Water One Month Before Test.

Condition of stave.	Load, in pounds, required to press a section, 3.35 in. long, of bands of various diameter, into the stave to a bearing of 60 and 90° of arc.								Average for all tests.
	0.193 in. Diameter.		0.226 in. Diameter.		0.497 in. Diameter.		0.625 in. Diameter.		
	60°	90°	60°	90°	60°	90°	60°	90°	
Staves containing Sapwood.									
Natural	166	371	218	453	507	1 096	608	1 334	594
Treated	171	414	250	518	620	1 338	756	1 663	716
Treated, in percentage of natural	103.1	111.6	114.7	114.4	122 3	122.1	124.4	124.7	120.5
Staves all Heartwood.									
Natural	196	460	267	574	592	1 291	714	1 572	708
Treated	210	531	332	713	752	1 596	880	1 929	868
Treated, in percentage of natural	107.1	115.5	124.4	124.2	127.0	123.6	123.2	122.7	122.6

After this treatment, the staves were free from excess of oil and easy to handle. They were then placed in a water tank with the untreated pieces, and all were soaked for about 30 days. Then both the natural and creosoted staves were subjected to a band pressure test. In making this test, four sizes of bands were used, as shown in Table 4. Each band was 3.35 in. long, and was pressed into the stave, for the entire length of the band, in a direction perpendicular to the grain of the wood, until it was embedded over an area equal to 60 and 90° of the arc. Figs. 8 and 9 illustrate the method of making the test. The loads required to cause this depth of compression are shown in Table 4. Each stave, natural and treated, was subjected to tests with bands of each size. The results show clearly that creosoted staves

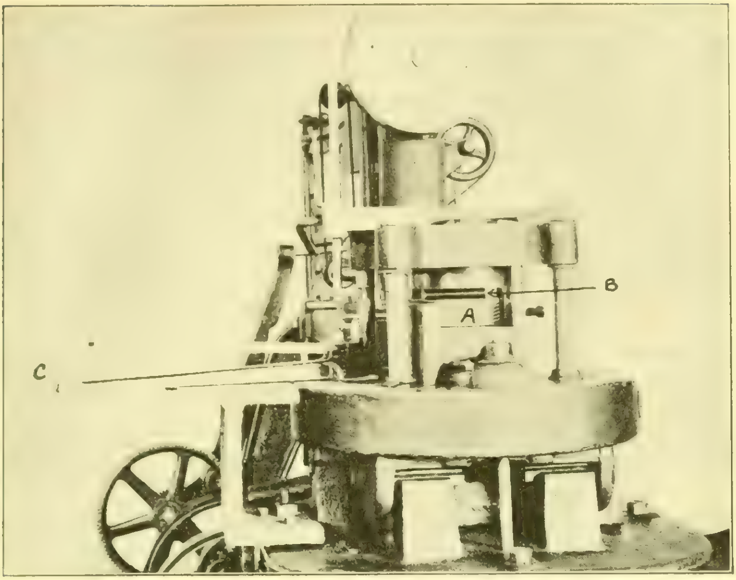


FIG. 8.—METHOD OF MAKING BAND PRESSURE TEST.
A.—STAVE UNDER TEST.
B.—BAND BEING PRESSED INTO STAVE.
C.—INSTRUMENT FOR READING COMPRESSION.

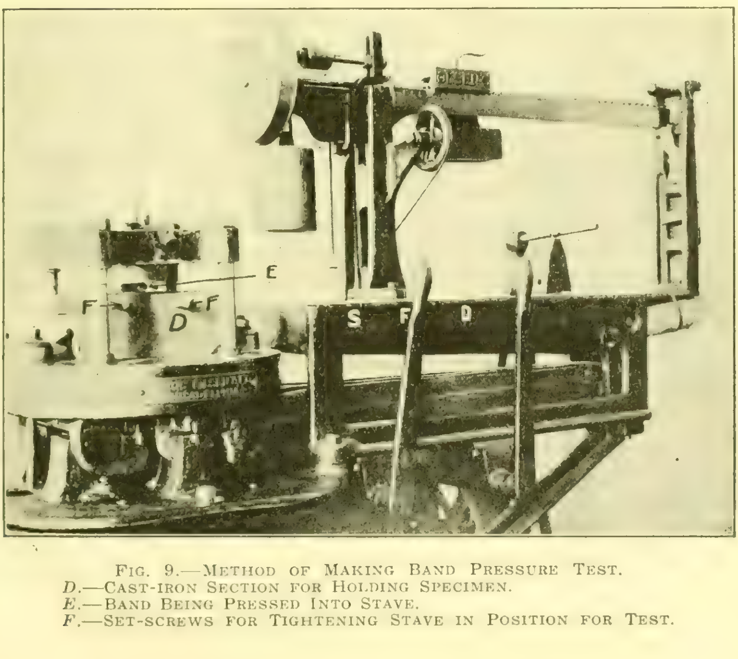
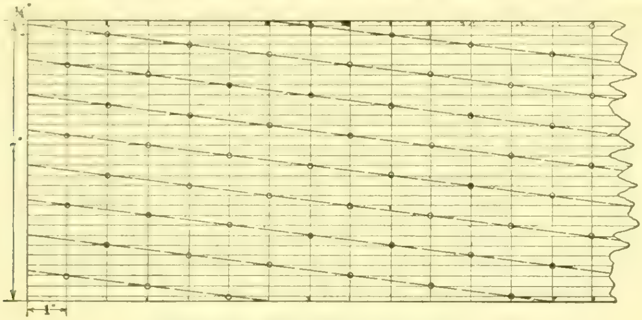


FIG. 9.—METHOD OF MAKING BAND PRESSURE TEST.
D.—CAST-IRON SECTION FOR HOLDING SPECIMEN.
E.—BAND BEING PRESSED INTO STAVE.
F.—SET-SCREWS FOR TIGHTENING STAVE IN POSITION FOR TEST.

of all heartwood as well as those of mixed heartwood and sap wood have strength values even greater than those obtained from the test of natural wood. The results are particularly significant, as the test approaches about as closely as possible the actual condition of staves in a pipe line in service. The creosoted staves uniformly show a slightly higher strength than the untreated staves tested under the same conditions. These results demonstrate clearly the fact that it is possible to creosote Douglas fir staves and retain all their original strength. They also indicate clearly the practicability of permitting sap wood in staves which are to be creosoted. In the staves tested, the penetration of the creosote oil in the sap wood was very complete in every case. Mr. Goss.

Recent experimental work conducted by the writer has also shown the practicability of perforating the outer surface of Douglas fir



ARRANGEMENT OF PERFORATIONS
WHICH WILL RESULT IN COMPLETE
PENETRATION OF CREOSOTE OIL
EQUAL TO DEPTH OF PERFORATIONS.
FIG. 10.

staves with small holes systematically spaced, as shown in Fig. 10. The resistance of the oil passing into the wood along the grain is practically nil compared to that found in forcing the oil in across the grain. The fine holes, through which the depth of penetration and the distribution of oil is controlled, are so small and spaced so regularly that they do not reduce the strength of the wood when tested in compression perpendicular to the grain. The result of these perforations is that the depth of penetration and the distribution of the oil can be thoroughly controlled—two very important factors in securing the greatest efficiency from creosoted timber.

Mr. Partridge does not discuss at any length the possibility of the entire elimination of decay in wood stave pipe by an efficient pressure treatment of the staves with coal-tar creosote. This subject should be thoroughly investigated at once, as the proper use of creosote promises to extend greatly the field of usefulness of such pipe, and give it a

Mr. Goss. reliability never before possessed. The writer is fully convinced of this, after having studied the subject very carefully. A large number of test results are available, and more should be obtained.

The final cost of a creosoted stave pipe line in place will be approximately 20% more than that of an untreated Douglas fir line, and about the same as that of an untreated redwood line. A number of large items remain constant, regardless of the cost of the staves. The steel bands, the excavation, the back-filling, engineering, overhead, transportation, and other costs remain constant regardless of whether or not the staves are creosoted. This added cost of 20% for a Douglas fir line may be assumed to buy a guaranty of greater permanency for any line, regardless of conditions under which it is built.

The writer recommends a thorough investigation by the Society of the possibilities of creosoted wood stave pipe. Creosote promises to revolutionize the use of wood stave pipe and put it even more firmly in a class with the most durable pipe known, and yet leave it with almost all its present advantages as to low first cost and low coefficient of friction. The writer has discussed this subject with Joseph Jacobs, L. J. Stannard, and Marvin Chase, Members, Am. Soc. C. E., the latter recently appointed Irrigation Engineer for the State of Washington, all of whom see great possibilities for the creosoted stave form of construction. Mr. Chase recently built approximately 8 200 ft. of 60 and 63-in. (inside diameter) creosoted wood pipe for the Wenatchee Reclamation District, in the Wenatchee Valley, Washington, and this line gives promise of most excellent results.

Before any standard specifications are adopted, the American Society of Civil Engineers should thoroughly investigate this subject through a committee capable of going deeply into the subject.

Mr. Nimmo. W. H. R. Nimmo,* Assoc. M. Am. Soc. C. E. (by letter).†—The writer, having had occasion to deal with numerous lines of wood pipe, especially of the machine-banded type, for town water supplies, has been much interested in this paper. Wood pipes as used in Tasmania, whether of the continuous-stave or machine-banded type, are almost always constructed of fir, known locally as Oregon fir. Although some engineers are fully aware of the superiority of redwood, yet, owing to its greater cost, pipes of this timber cannot be obtained from stock, and it is necessary to have them manufactured especially for each job. In the smaller sizes, up to about 6 in., redwood pipes cannot usually compete with cast iron or reinforced concrete. This State possesses some fine timbers, which may possibly prove superior to redwood, yet, at the present time, owing to lack of a sound forest policy, they cannot be obtained as cheaply as Oregon fir.

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* Hobart, Tasmania.

† Received by the Secretary August 20th, 1917.

Machine-banded pipes, after receiving a first coating of bituminous composition, are usually wound spirally with a strip of Hessian, and are then given a second coating of composition and rolled in sawdust. The Hessian covering and second coating increases the cost slightly, and is omitted in some cases. Mr. Nimmo.

If galvanized wire of good quality is used, a factor of safety of four against breaking is considered sufficient. In computing the stress on the winding, the question arises as to the exact diameter of the pipe to be used. In a wood pipe, which is saturated, the joints between the staves must contain water under pressure, and in designing such pipes, the writer assumes that the pressure varies uniformly from the full pressure at the inner surface to zero at the outer surface, and the mean diameter of the pipe is used in computing the stress in the wire. This is a matter of no importance in a large pipe, but, in small sizes, its effect on the size of the wire is appreciable. The writer has not seen the question dealt with by any authority.

In the case of continuous-stave pipe, the design of shoes has not always received sufficient attention. A type of shoe is sometimes used in which the tension of the two ends of the band are not in the same vertical plane, thus adding a horizontal bending stress to the direct tensile stress in the bands near the shoe.

A combination of inserted joint with a collar is now generally used on machine-banded pipes, and has been found to be fairly satisfactory.

The increase in shipping weight due to the coating may not be of great importance. Freight by sea is usually charged by measurement, and special rates by rail can frequently be obtained for complete carloads. The smaller freight and handling charges on wood pipe, as compared with other kinds of pipe, however, are often the determining factors in its selection.

In all specifications for machine-banded pipes a clause should be inserted requiring each pipe to be clearly branded with letters indicating the job, and figures indicating the head for which it is intended. The writer knows of cases where pipes intended to be laid near the intake of a line, under a low pressure, have been carelessly laid in places where the pressure was comparatively high, resulting in considerable trouble in maintenance.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

PAPERS AND DISCUSSIONS

OBSTRUCTION OF BRIDGE PIERS TO THE FLOW OF WATER

Discussion.*

BY MESSRS. A. J. WILEY, R. D. GOODRICH, E. W. LANE, MANSFIELD MERRIMAN, AND F. H. FRANKLAND.

A. J. WILEY,† M. AM. SOC. C. E. (by letter).‡—The formula proposed by the author appears to fit the conditions under which his experiments were made, but it seems to the writer that these conditions, in which the pier occupied nearly 25% of the channel, are not comparable to the case of the ordinary bridge pier. In the author's experiments, the standing wave around the nose of the pier, as shown by the photographs, extended across the entire channel, whereas, in the case of an ordinary bridge pier, which occupies a relatively small part of the channel, the standing wave would not materially affect the hydraulic conditions. Mr. Wiley.

In the author's formula for back-water:

$$H = \frac{Q^2}{2\,g \left[C\,W \left(H_2 - 0.3 \frac{V_2^2}{2\,g} \right) \right]^2} - \frac{1.8\,V_1^2}{2\,g} \quad \text{..........} \quad (1)$$

H = head caused by back-water,
C = a coefficient,
W = width between piers,
H_2 = normal depth below piers,
V_1 = velocity of approach, and,
V_2 = velocity of retreat, or normal velocity below piers.

* This discussion (of the paper by Floyd A. Nagler, Jun. Am. Soc. C. E., published in May, 1917, *Proceedings*, and presented at the meeting of September 5th, 1917), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† Boise, Idaho.

‡ Received by the Secretary, July 16th, 1917.

Mr. Wiley.

In the formula it is evident that the expression,

$$W\left(H_2 - 0.3\frac{V_2^2}{2g}\right),$$

is the cross-section in the minimum section between the piers, so that the author's formula may be written,

$$H = \frac{Q^2}{2g(CA)^2} - \frac{1.8\,V_1^2}{2g} \quad \text{.................... (2)}$$

Also,

$$\frac{Q}{A} = V_0$$

where A is the area and V_0 the velocity at the contracted section between the piers.

The formula thus becomes:

$$H = \frac{V_0^2}{C\,2g} - \frac{1.8\,V_1^2}{2g} \quad \text{.................... (3)}$$

The author's coefficient, C, varies from 0.861, for a pier with both ends square, to 0.939 for one with nose and tail tapered. It appears, therefore, that H would be zero, and there would be no back-water effect unless the contraction caused by the pier was such that the square of the velocity between the pier would be 1.55 times the square of the velocity of approach for a square-ended pier, or 1.68 times for a taper-ended pier. Of course, there must be some back-water effect, no matter how slight the contraction caused by the pier, and, unless the writer is wrong in his interpretation of the author's formula, it cannot be of general application.

In one case observed by the writer, a concrete-lined canal is crossed by a wagon bridge with square-ended piers, 8 in. thick, and 14.32 ft. between centers. The following are the values of the different elements in this case:

$Q = 472$ sec-ft., between adjacent piers,
$W = 13.65$
$H_2 = 7.82$
$V_2 = 4.22$

Levels taken for a distance of 100 ft. above and below the bridge, on each side of the canal, which was 87 ft. wide at the water surface, and, corrected for the normal fall in the water surface, gave a back-water head of 0.077 ft. Levels taken in the center immediately above and below the bridge gave a back-water head of 0.15 ft. The author's formula gives the back-water head for these conditions as — 0.10 ft.

It will be observed that the author's formula, as reduced to Equation (3):

$$H = \frac{V_0^2}{C\,2g} - \frac{1.8\,V_1^2}{2g}$$

is the same as that of D'Aubuisson given by the author as: Mr Wiley

$$Y = \frac{V_2^2 - V_0^2}{2\,g},$$

or, changing the notation to conform with that used in the author's formula,

$$H = \frac{V_0^2 - V_1^2}{2\,g},$$

except for the application of the coefficient, 1.8, to the velocity head above the piers and the use of coefficient, C.

The writer is of the opinion that this formula, as modified by Eytelwein and quoted by the author, is an entirely rational expression for the back-water, and will fit ordinary conditions better than that proposed by the author.

Changing the modified D'Aubuisson-Eytelwein formula into the terms used in the author's formula, it is as follows:

$$H = \frac{V_1^2}{2\,g}\left(\frac{W_1^2}{C^2\,W^2} - \frac{H_2^2}{(H_2 + H)^2}\right)$$

in which W_1 is the width of approach.

The application of this equation to the case of the piers in the concrete-lined canal previously mentioned, gives a back-water effect of 0.130, which is probably as close to the actual back-water as the average head of 0.077 ft., or the center measurement of 0.15 ft.

The author's method of determining the depth at the minimum section between the piers is to assume that it will be less than that below the piers by the amount of the recovered velocity head, which, of course, is correct; but this percentage of recovery would vary for different conditions, and would have to be assumed for any other condition than that existing in the experiments.

In the Eytelwein formula, the depth between the piers is considered the same as that in the normal channel below the piers. This neglects the recovery of any part of the velocity head, and gives a slightly larger contracted area and a smaller velocity than actually exists. It is not practicable to design piers with the proper curves to recover all the velocity head, and the error in neglecting the recovered velocity head in calculating the velocity between the piers is not appreciable. The coefficient of contraction in the Eytelwein formula can easily be estimated by comparison with the coefficient of a submerged orifice with similar characteristics. The two coefficients in the author's formula would have to be assumed for any conditions different from those of the experiments, and there seems to be no criterion available in selecting these coefficients. It is the writer's belief that, though credit is due the author for the thoroughness of his experiments, the formula deduced therefrom, if applied to other conditions, may lead to entirely wrong conclusions.

Mr. Goodrich.

R. D. Goodrich,* M. Am. Soc. C. E. (by letter).†—The writer was engaged in a special study of the effects of an encroachment of a proposed building on a river, at a bridge on the main street of a certain city, when the *Proceedings*,‡ containing Mr. Nagler's paper with its discussion of existing bridge pier formulas was issued. Consequently, the results of his experiments and his new formula were most opportune, and were studied with the greatest interest. After some consideration of the theory used in the development of the formula, and some comparison of the results obtained by the various formulas given, the writer decided to use it in the preliminary computation of the back-water which might be expected from the proposed encroachment.

The discharge of the river at the highest stage known was taken from United States Geological Survey reports, and a probable mean velocity of 4 ft. per sec. was determined. The back-water caused by the bridge, with its one center pier, was then determined, the channel having a clear width of about 206 ft. The back-water, with the width of the channel reduced to 170 ft. was then computed, and the difference of 3 in. in the two cases was used as the height of the back-water to be expected from the construction of the building. By assuming the velocity of approach to be the same in both cases, this factor can be eliminated before substituting numerical values, and the increase in back-water, due to an encroachment at one side of the channel, can be found directly.

Having completed these preliminary calculations, the city attorneys drew up their bill of complaint, and a temporary restraining order was granted to prevent further construction. Among other grounds for injunction, this additional 3 in. of back-water was set up as one cause of complaint. The work of preparing the case for an early hearing was then taken up in earnest, but a great disappointment was encountered.

Surveys were made of the river immediately above and below the bridge in question, new data were collected, and all the computations were reviewed. As the revision progressed, it was found that the velocity in the channel above the bridge would be more than 5 ft. per sec. With this velocity, however, the formula gave a negative back-water. The computations were reviewed again and again, but the formula failed entirely just outside the actual range of Mr. Nagler's experiments.

At this writing (July, 1917), the final hearing is being had, and the testimony of a well-known and experienced hydraulic engineer, employed by the city, shows that about 1 in. of additional back-water may be expected to result from the erection of the building. The

* Lansing, Mich.

† Received by the Secretary, July 28th, 1917.

‡ *Proceedings*, Am. Soc. C. E., May, 1917.

Mr. Goodrich.

explanation to the Court of the disappearance of the other 2 in. of back-water is not anticipated with any great pleasure, but it will be easier than to tell how the water is piled up higher below the bridge than above it. The lesson on the use of untried formulas is so obvious that it is passed on for whatever it may be worth. It is fortunate for the writer that the character and limitations of the author's new formula were discovered previous to the hearing, and a much more embarrassing situation thus avoided.

A full publication of Mr. Nagler's experiments would have been of the greatest value in studying this problem, and it is to be regretted that such a paper as this is not published with the tables in full for the benefit of the majority of the members of the Society who reside outside of New York City and cannot avail themselves of the privileges of the Society Library. The writer is well aware of the great care with which the author conducted his experiments, and it is to be hoped that, with further work along the same line, a more logical formula will be evolved in time.

The following is a statement of the conditions in the special problem which led to this discussion:

Discharge, $Q = 19\,500$ sec-ft.;

Net width at bridge, $W = 203$ ft.;

Mean depth at bridge, $H_2 = 16.4$ ft.;

$C = 0.894$ (using for the pier the D nose and the A tail as described in the paper);

Area of channel above bridge $= 3\,680$ sq. ft.;

$$V = \frac{19\,500}{3\,680} = 5.3 \text{ ft. per sec.};$$

$$h = \frac{V^2}{2\,g} = 0.4367,\ 1.8\,h = 0.786,\ 0.3\,h = 0.131.$$

Assume the velocity of retreat to be equal to the velocity of approach; then, from the formula for Q, on page 841,*

$$H = \frac{Q^2}{2\,g\,C^2\,W^2\,(H_2 - 0.3\,h)^2} - 1.8\,h$$

$$= \frac{19\,500^2}{64.4 \times 0.894^2 \times 203^2\,(16.4 - 0.13)^2} - 0.786$$

$$= 0.686 - 0.786 = -0.100.$$

Mr. Lane.

E. W. LANE,† JUN. AM. SOC. C. E. (by letter).‡—The writer has studied this paper with a great deal of interest, as he has been carrying on a series of experiments along a similar line for several years. The data presented are of great value, as they constitute practically the only reliable information on the subject which has been pub-

* *Proceedings*, Am. Soc. C. E., May, 1917.

† Rantoul, Ill.

‡ Received by the Secretary, August 25th, 1917.

Mr. Lane. lished, and Mr. Nagler is to be commended for the service which he has rendered in making it available to engineers. Some of the conclusions are somewhat unexpected, particularly those regarding the efficiency of the semicircular ends (Type *B*), the 90° point up stream (Type *C*), and the effect of the shape of the down-stream end of the pier. In general, the writer agrees with the statements in the paper, but on several important points his observations and experiments have led to conclusions differing somewhat from those of the author.

Experiments on models, such as those described in this paper, are valuable, as they often point out the solution of a problem when an investigation on full-sized apparatus is impossible. Because of the many factors involved in most hydraulic phenomena, however, it is unsafe to apply the results of such experiments directly to actual conditions. Reliable conclusions can only be reached by making a careful analysis of the results of the experiments, to arrive at the underlying causes, and, from a knowledge of these, determining how far these results may be modified by the larger scale of the actual case.

Although the title of the paper is "Obstruction of Bridge Piers to the Flow of Water", and the formula derived by the author gives the height of the back-water caused by them, the writer believes that the increased height of the water surface above a bridge may be influenced by other factors than the size and shape of the piers. The principal causes of back-water are:

(1).—The reduction of the cross-sectional area of the stream as it flows under the bridge;

(2).—The effect of the nose of the pier;

(3).—The effect of the tail of the pier;

(4).—Friction, both of the water against the bottom and sides of the channel and internal frictions in the water itself.

The reduction of the cross-sectional area of the stream as it flows under the bridge is usually the most important factor causing back-water. In this reduction, the piers often play a minor part, their effect sometimes being absent altogether. Perhaps the most frequent cause of serious back-water is the construction of highways or railroads across the bottom land on high fills, compelling the water, which, in times of flood, formerly flowed over the entire valley, to pass through the comparatively narrow opening at the bridge. This was the condition in the Lackawanna *vs.* Erie case cited by the author, and in several other instances which have come to the writer's attention.*

* Report of the Chief Engineer of the Miami Conservancy District, 1916, Vol. I, pp. 67–71. A Report to the Mayor and City Council on Flood Prevention for the City of Columbus, Ohio, Alvord and Burdick, September 15th, 1913, pp. 80–89.

Mr. Lane.

When the cross-section of a stream is thus reduced, in order to flow through the smaller area, the velocity of the water must be increased, and a backing up above the opening occurs until the water reaches an elevation sufficient to create this higher velocity. The change from elevation head to velocity head takes place according to the well-known theorem of Bernouilli.

The action of the nose of the pier is to cause the water to flow through an area smaller than the actual area between the piers. One who has closely observed the action of a bridge pier in times of high water has no doubt noticed that a considerable space adjoining the pier is filled with eddies, in which the motion of the water has only a small component in the direction of the flow of the stream. This effect is particularly noticeable where the nose of the pier is poorly designed, or where the piers are not properly set with reference to the direction of the current. In many cases the abutments have a similar action, particularly if they project out considerably into the stream channel. Because of the presence of this eddy-filled space, the area through which the flowing water must pass is less than the actual cross-section of the water under the bridge, and the velocity of the water, therefore, must be correspondingly greater, necessitating a greater heading up above the bridge. As the height varies as the square of the velocity of the water, it is evident that the effect of the nose of the pier may be an important factor.

A striking confirmation of this explanation of the effect of the nose of the pier is given in Fig. 20, showing the stream lines about the model piers. In all cases, with two exceptions, for piers having tails of the same type, the coefficients grow less as the area shown by the sketch to be occupied by eddies increases. For example, of the piers having tails of the *B* type, Fig. 20 shows that when arranged in decreasing order of area occupied by eddies, they are as follows: *AB, CB, DB, BB,* or *FB*. The lines about *AB* are not shown, but they may be judged from those about *AA*. This order is the same as when these piers are arranged in increasing order of coefficients. The only exceptions to this rule are Models *EA* and *FA*, which seem to be the same as the "inconsistencies" mentioned by the author.

The third factor, the effect of the tail of the pier, is more difficult to analyze. Perhaps the most common explanation would be that the higher coefficients of a certain pier form are due to the recovery of head resulting from the gradual enlargement of the cross-section of the stream. Although the effect may be due in some measure to this cause, the writer believes that the action of the tail of the pier is similar to that of the nose, namely, its influence on the effective cross-section of the stream. Below a bridge pier, in times of high water, eddies or vortices are formed by the action of the flowing

Mr. Lane. water on the comparatively still water in the space directly behind the piers. The vortices on the two sides of the pier diverge as they pass down stream, in extreme cases meeting those from adjacent piers some distance below. The action of these eddies further reduces the effective area of the bridge opening and increases the back-water. This seems to be the author's conclusion from the statement on page 816*: "The pier with a tail which follows the diverging stream lines most closely, reducing the volume of eddying water to a minimum, should be most efficient at the down-stream end." Diagrams made during the flood of the Seine in 1910, showing the formation of these eddies by the bridges of Paris, have been published.†

That the effect of the tail of the pier is of much less importance than that of the nose is shown by a comparison of the coefficients in Fig. 8. The difference between the coefficients of the most and least efficient piers of any set having tails of the same type, ranges from 0.061 to 0.079, with an average of 0.070; and for the same shaped nose and different tail shapes, it varies from 0.016 to 0.040, or, neglecting the apparently inconsistent results for Piers *EA* and *FA*, it ranges from 0.016 to 0.023, with an average of 0.020. The relative importance of the nose and tail of the pier, therefore, is about as 7 to 2.

The effect of friction loss has been little discussed by the author, because, in his experiments, it was negligible. In actual cases of back-water, however, this may become an important factor. The piers of bridges are often protected from undermining by large piles of loose stone around the base, or the bottom of the stream may be rough or filled with piling. In connection with the flood prevention work of the Miami Conservancy District,‡ the writer had occasion to measure the flow through the contraction of a river channel caused by the construction of a bridge. As there were no piers in the stream, the entire change of elevation head, not accounted for by the increased velocity, was lost in friction. The high velocity of the water had washed away the finer material, leaving the bottom covered with rocks, averaging, perhaps, 9 in. in diameter. The only condition tending toward high friction loss in this measurement, which would probably not be present in most cases of back-water, was the comparatively small depth of the flowing stream, which varied from 4 ft. above the bridge, to 2 ft. in the most contracted section. Of the total drop of 1.36 ft., 0.43 ft. was changed into velocity and the remaining 0.93 ft. was lost in

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* *Proceedings*, Am. Soc. C. E., May, 1917.

† *Annales des Ponts et Chaussées*, July–August, 1911, Ninth Ser., Vol. IV, Part IV, pp. 11–53.

‡ A complete description of this measurement will be published in the Technical Report of the Miami Conservancy District, Part IV, "Calculation of Flow in Open Channels."

friction. It is evident, therefore, that in some cases friction may be of considerable importance. Mr. Lane.

If the foregoing analysis correctly represents the effect of the factors causing back-water at bridges, the rational formula for this phenomena would be developed as follows: Suppose that the first of the four causes is the only one acting, and that there is no recovery of head below the piers. The formula, using the author's notation, would be, by Bernouilli's theorem:

$$h + y + \frac{v_0^2}{2g} = h + \frac{v_2^2}{2g},$$

or,

$$y = \frac{v_2^2 - v_0^2}{2g} = \frac{V_0^2}{2g}\left[\left(\frac{W(h+y)}{wh}\right)^2 - 1\right].$$

If we consider also the effect of the nose and tail of the pier, as described above, the effective area between the piers becomes $m\,w\,h$, where m is the ratio of the effective to the actual width between the piers, and

$$y = \frac{v_0^2}{2g}\left[\left(\frac{W(h+y)}{m\,w\,h}\right)^2 - 1\right]$$

which, expressed in the terms of Q, is

$$y = \frac{Q^2}{2g}\left[\frac{1}{(m\,w\,h)^2} - \frac{1}{W\;(h+y)^2}\right].$$

This is the form ascribed to Debauve, although the writer, as will be shown later, believes that it was first derived by d'Aubuisson. The writer does not agree that this is the same as the formula accredited to Eytelwein, as it gives results $\left(\frac{h+y}{h}\right)^2$ times as large. In actual cases, the channel is not rectangular, as is assumed in this notation, and a more practical formula results by replacing $W\;(h+y)$ by A_0, the cross-sectional area of the stream above the bridge, and $w\,h$ by A_2, the area between the piers, making the foregoing formula

$$y = \frac{Q^2}{2g}\left(\frac{1}{m^2 A_2^2} - \frac{1}{A_0^2}\right).$$

In this formula, as the coefficient, m, is the ratio of the effective area to the actual area under the piers, other things being equal, the opening having the greatest number of piers would have the greatest part of the area filled with eddies, and, therefore, the lowest coefficient. In the same way, within certain limits, the wider the bridge pier, the wider would be the eddy-filled area about it, and the lower would be the coefficient. As the author's formula, as will be shown later, differs only slightly from that just given, the writer does not agree with his conclusion that the size and spacing of the piers would

Mr. Lane. have no effect on the coefficient of discharge of bridge openings. The experimental evidence offered does not necessarily lead to that conclusion. Although the coefficient of a small orifice, or even one 4 ft. square, may be about 0.62, one would hardly expect that, with the same head, the width at the *vena contracta* of a stream, issuing from an orifice 100 ft. wide, would be only 62 ft. The action would probably be more like that of a weir with end contractions, where the effect of the contraction at each end reduces the effective length by 0.1 H. In the case of the trash racks cited by the author, to give the same "total area of obstruction", with twice the number of bars, requires that the width of each bar be just half as great; and the loss occasioned by having twice the number of bars, may be offset by their being only half as wide.

The modification of the formula developed previously to take account of friction must necessarily be largely a matter of judgment, as so little is known of its influence that exact analysis is impossible. An interesting method of introducing this factor was developed by Ivan E. Houk, Assoc. M. Am. Soc. C. E., in connection with the studies of the Dayton flood.* This method was used in determining the discharge of the Miami River and its tributaries from measurements of the drop at bridge openings, but is not so readily applied to the reverse operation of finding the height of back-water resulting from a given discharge. Perhaps the most practical way of taking account of friction would be to decrease the coefficient, m, by an amount dictated by judgment, based on the conditions existing at the particular opening under consideration.

The formula developed previously may also be expressed in the form,

$$Q = C\,A_2\sqrt{2\,g\,\left(H + \frac{v_0^{\,2}}{2\,g}\right)},$$

where H is the drop and corresponds to the y of the preceding formula, and the symbol, C, is used to express the contraction coefficient in place of m. It will be seen, therefore, that the discovery that the "discharge might more correctly be represented as that through an orifice of a sectional area equal to the minimum stream width multiplied by the minimum stream depth, under a head, H, equal to the back-water caused by the bridge pier (corrected for velocity of approach)", is not new, having been known not only by Unwin and Frizell, but was predicted by d'Aubuisson (or Debauve), was published by d'Aubuisson in his "Treatise on Hydraulics" in 1840, and is a direct application of the Theorem of Bernouilli, which was proposed nearly 200 years ago.

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* Report of the Chief Engineer of the Miami Conservancy District, 1916, Vol. I, pp. 67–71. The Official Plan of the Miami Conservancy District, Vol. I, pp. 67–71.

Mr. Lane.

The writer agrees that this proposition is the rational basis for the back-water formula, and that the formulas of Weisbach and Merriman, when applied to bridge openings, violate "a very fundamental principle of hydraulics." In practical cases, however, that part of the discharge which, according to these formulas, is assumed to flow over a weir, is usually relatively small, and does not introduce great error in the result. It is interesting to note that, in the tenth edition of his "Treatise on Hydraulics," Professor Merriman has adopted the form ascribed by the author to Eytelwein, which the author believes, "in all probability, is the most correct formula which has been derived, taking the preceding theory as a basis." The writer believes, as will be shown later, that this formula is also erroneous. Experiments made by him, which have not yet been completed, indicate that, for certain conditions of flow through contracted openings, the Weisbach formula is not only empirically, but also theoretically, correct. These conditions, however, do not occur in the ordinary case of back-water at bridges.

The formula developed previously differs very little from the author's final form, as it can be expressed by the following equation:

$$Q = C\,W\,\sqrt{2\,g}\left(H - \alpha\,\frac{v_2^{\,2}}{2\,g}\right)\sqrt{H + \beta\,\frac{v_1^{\,2}}{2\,g}}$$

where α and β are zero and 1.0, in place of the 0.3 and 1.8, respectively, adopted by the author. The justification for the introduction of the term, $\alpha\,\frac{v_2^{\,2}}{2\,g}$, seems to be (from Fig. 19) that there is a rise in the water surface down stream from the piers. That this occurred in the experimental flume, the writer has no doubt, but he does not believe that it is representative of the usual conditions of back-water. It should be noticed that the waterway on each side of the model pier had a width of only about 1.6 times the width of the pier. The wave formations about the pier, therefore, would occupy a much greater relative space in this flume than in actual conditions, where the corresponding space would probably be about six to ten times the width of the pier, and a rise, which in actual conditions might take place only in the immediate vicinity of the pier, and not affect appreciably the greater part of the flow, in this narrow flume would occupy the whole width.

The only information bearing on the question of the rise occurring below bridge openings, of which the writer has knowledge, was that collected under the direction of Mr. Ivan E. Houk, in connection with the investigation of the Dayton flood by the Morgan Engineering Company. Detailed surveys were made at a number of bridge openings, and in only one case was there any evidence that the elevation

Mr. Lane. of the water surface a short distance down stream from the opening was higher than the lowest point of the water surface in the opening. Practically, however, the term, $\alpha \frac{v_2^2}{2g}$, is usually a very small correction, for, even with v_2 equal to 16 ft. per sec., this correction would be only 1.2 ft., and, as it is subtracted from H_2, which is usually comparatively large, it has relatively little effect. The effect of the $1.8 \frac{v_1^2}{2g}$ is greater, as it is added to the observed head, which is never more than a few feet. On the other hand, the discharge is a function of $\sqrt{H + 1.8 \frac{v_1^2}{2g}}$, and the importance of the velocity of approach correction, therefore, is reduced.

From his search of the literature dealing with the flow of water through contractions in a channel, the writer has not arrived at quite the same results as the author regarding the origin of the formulas quoted. Perhaps the author had access to information not available to the writer, but, inasmuch as certain statements already printed in the publications of this Society differ somewhat from those in this paper, it may be well to have the origin of the various formulas established. The late William R. Hutton, M. Am. Soc. C. E., in his paper "On the Determination of the Flood Discharge of Rivers and of the Back-water Caused by Contractions,"* writes as follows:

"To determine the increased rise which would be caused by the contraction of the water-way by the works of the Lackawanna Company, two formulas are used, which furnish very different results. The first is that of Eytelwein, the second is quoted from Debauve (*Manuel de l'Ingénieur des Ponts et Chaussées. Ponts et Maçonnerie*), but had long before been given by d'Aubuisson. The great discrepancy in the results given by these two formulas (the rise y by the Eytelwein formula would be 2.3 ft., by that of d'Aubuisson 4.3 ft.), both of them founded upon the same general principle, has suggested an examination of the process by which they were constructed, which seems to be worth recording.

"Dubuat, with whom the method originated, assumes that there will be no considerable change of velocity in the section above the contraction. Consequently the velocity in the contraction will be to that above it, inversely as the sections, $v_{11} = \frac{v_0 W}{w}$, and the difference of the heights to which these velocities are due will be

$$y = \frac{v_{11}^2 - v_0^2}{2g} = \frac{v_0^2 W^2}{2g w^2} - \frac{v_0^2}{2g} = \frac{v_0^2}{2g} \left(\frac{W^2}{w^2} - 1\right);$$

* *Transactions*, Am. Soc. C. E., Vol. XI, p. 239.

and the coefficient which is to compensate for the contraction of the stream in the narrow water-way is applied to the resulting height, so that in its final form we have $y = \frac{v_0^2}{m^2\,2\,g}\left(\frac{W^2}{w^2} - 1\right)$. Mr. Lane.

"In the first edition (1801) of his *Handbuch der Mechanik und der Hydraulik,* Eytelwein follows Dubuat in neglecting the diminution of the velocity above the contraction, caused by the raising of the surface, but he applies the correction for contraction to the narrow section only. Thus making

$$y = \frac{1}{m^2}\left(\frac{v_0\,W}{w}\right)^2 - \frac{v_0^2}{2\,g} \text{ or } \frac{v_0^2}{m^2}\left(\frac{W^2}{w^2} - \frac{1}{2\,g}\right);$$

m being in this case $\sqrt{2\,g} \times 0.95$.

"In the edition of 1843 of the same work he makes

$$v_{11} = \frac{v_0\,Q}{w\,(h + y)} = \frac{v_0\,W\,h}{w\,(h + y)} \text{ and } y = \frac{v_0^2}{m^2}\left(\frac{W\,h}{w\,(h + y)}\right)^2 - \frac{v_0^2}{m^2}$$

or

$$\frac{v_0^2}{m^2}\left(\frac{W^2\,h^2}{w^2\,(h + y)^2} - 1\right),$$

as quoted by General Gilmore.

"It will be observed that v_0 being original mean velocity, v_{11} (that between the piers) is not $\frac{Q}{w\,(h + y)}$ but simply $\frac{Q}{w\,h} = \frac{v_0\,W\,h}{w\,h} = \frac{v_0\,W}{w}$. Again, as the second member of the primary equation is the head due to the velocity above the contraction, it is equal to $v^2\left(\frac{W\,h}{W\,(h + y)}\right)^2 = v_0\frac{h^2}{(h + y)^2}$, which does not vary with the coefficient of contraction. Substituting we have $y = \frac{v_0^2}{2\,g}\left(\frac{W^2}{m^2\,w^2} - \frac{h^2}{(h + y)^2}\right)$, m being the usual coefficient of contraction applicable to the character of the entrance to the contraction. This form involves smaller numbers than the formula quoted by General Gilmore from Debauve, but it is substantially the same."

From this quotation it seems that the formula ascribed to d'Aubuisson, according to Mr. Hutton, was proposed by Dubuat, and that ascribed to Eytelwein is not the same as either of those quoted by Mr. Hutton from Eytelwein's "Handbuch der Mechanik und der Hydraulik", but is a modification of them which was proposed by Mr. Hutton, and which the writer believes is based on erroneous assumptions.

The following is quoted from "A Treatise on Hydraulics," by J. F. d'Aubuisson, published in 1840, and translated by Joseph Bennett in 1852, back-water produced by contracting the water-way, page 188:

Mr. Lane.

"The height of this fall will also be given by the equation

$$p' = \frac{v^2}{2\,g} - \frac{v_0^{\,2}}{2\,g} = \frac{Q^2}{2\,g}\left(\frac{1}{s^2} - \frac{1}{s_0^{\,2}}\right).$$

Let x be the height of fall, L the mean breadth of the stream above the contracted space, l the width of the contracted part, and h, the depth of water in that part; its section, s, will be $l\ h$, or rather $m\ l\ h$, m being the coefficient of contraction at the entrance; for the section, s_0, of the current immediately above the fall, we have $L\ (h + x)$, $h + x$ being the depth of water there and L the breadth. Thus, observing that x is the slope designated above by p' we shall have

$$x = \frac{Q^2}{2\,g}\left(\frac{1}{m^2\,l^2\,h^2} - \frac{1}{L^2\,(h + x)^2}\right)."$$

The writer was not able to determine whether the work of Debauve antedates that of d'Aubuisson, but it is evident that they both used the same form, and that Mr. Hutton believed that d'Aubuisson was the originator of it.

A close examination of the reasoning of Mr. Hutton, in which he develops the formula,

$$y = \frac{v_0^{\,2}}{2\,g}\left(\frac{W^2}{m^2\,w^2} - \frac{h^2}{(h + y)^2}\right)$$

will show its incorrectness, from a theoretical standpoint. According to Mr. Hutton, "It will be observed that v_0 being the original mean velocity, v_2 (that between the piers) is not $\frac{Q}{w\ (h + y)}$ but simply

$$\frac{Q}{w\ \ h} = \frac{v_0\ W\ h}{w\ \ H} = \frac{v_0\ W}{w}."$$

The equalities given are erroneous, as $\frac{Q}{w\ \ h}$ is not equal to $\frac{v_0\ \ W\ h}{w\ h}$, but rather to $\frac{v_0\,W\,(h + y)}{w\,h}$, which is $\frac{h + y}{h}$ times as large as $\frac{v_0\ W\ h}{w\ h}$, and, therefore, accounts for the formula derived by Mr. Hutton and ascribed to Eytelwein, giving results $\frac{h^2}{(h + y)^2}$ times as large as that of Debauve (or d'Aubuisson), as previously mentioned.

In conclusion, though the writer does not believe that it has been demonstrated that all the existing formulas are erroneous, nor that the author's formula is "far more adequate and simple than any of the existing formulas", as it differs from one of the existing forms only by the addition of the empirical corrections, $0.3\,\frac{v_2^{\,2}}{2\,g}$ and $1.8\,\frac{v_1^{\,2}}{2\,g}$, the author has undoubtedly pointed out the true action of the water

in flowing through a bridge opening, and it is to be hoped that the formulas based on the combined weir and orifice theory, now so commonly used, will be rapidly discarded. It is doubtful if the results obtained from such small-scale models justify the adoption of the corrections mentioned above, when applied to actual cases of back-water, and the formula of Debauve (or d'Aubuisson), when expressed in terms of the area of the stream above and between the piers, is much more simple and practical. The effect of these corrections in most actual cases of back-water, however, will be so slight that the author's coefficients probably could be used with either formula, within the limit of accuracy which it is possible to obtain in practice. These coefficients supply a long felt need, as they offer a convenient index to the efficiency of the various types of piers, which will be of great value to designers of bridges. Although the experiments cover most practical forms of piers, a number of points, such as the effect of the size and spacing of the piers, were not determined, and it is hoped that these will be investigated in the near future. Mr. Lane.

MANSFIELD MERRIMAN,* M. AM. SOC. C. E. (by letter).†—The conclusions derived by the author regarding the relative efficiencies of piers of different forms are interesting and valuable. Although most of them have heretofore been known in a general way, this seems to be the only series of experiments which furnishes numerical comparisons, and hence the author has rendered a distinct service to hydraulic and bridge engineers. Mr. Merriman.

Fig. 19 is a good representation of the profile of the stream at the side of a pier; here, however, the hump at the nose of the pier and the depression along the side are local phenomena which are not seen at a distance of 10 or 15 ft. from the side of a common bridge pier. For two piers, 100 ft. or more apart, the profile of the stream surface half way between them is observed to be a straight line, and numerous other profiles do not resemble Fig. 19. Fig. 13, which the author criticizes, was not intended to represent a profile of the stream at the side of a pier, as the pier lines are drawn broken; it was intended, however, to represent approximately an average of all the profiles between two piers. It appears then, as the author admits, that there is some justification for the formula at the bottom of page 832,‡ although, like many hydraulic formulas, it cannot be regarded as theoretically perfect. That formula, however, is an awkward one from which to compute values of the back-water rise, H, and, therefore, it is to be desired that a more satisfactory one may be established.

* New York City.

† Received by the Secretary, September 5th, 1917.

‡ *Proceedings*, Am. Soc. C. E., May, 1917.

Mr. Merriman. The writer is inclined to the opinion that the formulas given on page 839* contain defects which do not justify their universal application, but he is very glad that the author has deduced such an extended series of coefficients for them. The main use of these formulas is, of course, for the computation of values of the back-water rise, H, from given values of Q, W, and H_2. Had the author given a numerical example showing how this may be done, it would have added to the interest of his valuable paper.

Mr. Frankland. F. H. FRANKLAND,† ASSOC. M. AM. SOC. C. E.—The author states that there are very few experimental data on the subject of the obstruction of bridge piers to the flow of water.

As evidence which would seem to indicate that the mathematical solutions of the problem and existing formulas are unreliable, attention is called to the writings of M. Bossut, the well-known French engineer, wherein his mathematical analysis of the problem led him to conclude that the pier nose should be a right-angled triangle in section. On the other hand, M. Dubuat, in his "Principles of Hydraulics," gives a mathematical solution which suggests a section having convex curves for the pier-nose faces.

Cresy's well-known experiments on the shape of piers were made many years ago, with various small models, about 15 cm. thick, and placed in a canal between vertical stop-boards which controlled the depth of water. Cresy found that the elliptical section gave better results than any other form, inasmuch as the resistance to flow was less and the stream contraction was a minimum. The worst results, as indicated by these experiments, were shown by a model with concave-faced pier noses.‡

Further, to illustrate the fact that previously existing formulas for pier design, in section, are misleading, the speaker has been told by J. A. L. Waddell, M. Am. Soc. C. E., that some years ago, during a controversy before the Secretary of War regarding the then existing Kansas City Southern Railroad Company's bridge at Ohio Avenue, over the Kaw River, at Kansas City, a certain German engineer testified that his mathematical investigations showed that these bridge piers would offer such obstruction to the stream flow as to cause a difference in water level between the up-stream and down-stream sides of the bridge, at high-water stage, of 11 in. Dr. Waddell stated positively that the obstruction would be negligible. Careful investigations made later by the Railroad Company confirmed this statement. In fact, the difference in elevation between the water on the up-stream and down-stream sides of the bridge is barely measurable.

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* *Proceedings*, Am. Soc. C. E., May, 1917.

† New York City.

‡ A résumé of these experiments is given in "Sub-Aqueous Foundations," by Charles Evan Fowler, M. Am. Soc. C. E.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

THE WATER SUPPLY OF PARKERSBURG, W. VA.

Discussion.*

By William M. Hall, M. Am. Soc. C. E.†

William M. Hall,‡ M. Am. Soc. C. E. (by letter).§—The writer is gratified with the extent of the discussion, and wishes to express his appreciation to each of those who have contributed to it. Mr. Hall.

The preliminary investigations for a new water supply for Parkersburg were commenced by Senator Camden—at his own expense—as early as about 1903, and were continued from time to time until Mr. Knowles was engaged in 1908. During this period he conferred with the writer many times. As stated by Mr. Fuller, two factions of the citizens were developed, one led by Senator Camden favoring a well supply, and the other led by the mayor of the city at that time, favoring a mechanical filtration system. Senator Camden spent several thousand dollars of his personal funds in extending the information as to the available natural supply obtainable from wells, and, besides, devoted much of his time to gathering information, locally and in the Ohio Valley, relative to the subject. He also presented the city with a tract of 43 acres for water-works purposes, but this was lost by the failure to adopt the well system.

The mayor in office at that period was skillful enough to use the subject so as to obtain two elections therefrom, but, on the third, it resulted not only in his complete downfall, but also in changing the three-headed form of city government—consisting of a council, board of affairs, and mayor—to one of simply a mayor and four commissioners working as a single body.

Mr. Johnson and others who wish for more data on the investigations by Mr. Knowles and Messrs. Fuertes and Fuller, in reference

* Discussion of the paper by William M. Hall, M. Am. Soc. C. E., continued from August, 1917, *Proceedings*.

† Author's closure.

‡ Parkersburg, W. Va.

§ Received by the Secretary, September 8th, 1917.

Mr. Hall. to the natural available well supply on the Camden Farm, the capacity of the wells, the comparison of the several waters chemically, the bacteria, *B. coli,* and bacteria count, should obtain printed copies of these reports from the city clerk; they contain many valuable data, too extensive for this paper.

A copy of the test made under the direction of Mayo Tolman, Jun. Am. Soc. C. E., Chief Engineer of the State Board of Health, for the fiscal year past, as prepared by him and given out for publication under date of August 13th, 1917, is given in Table 8.

In a further report, in reference to this supply, Mr. Tolman says:

"The Parkersburg water supply is undoubtedly safe for consumption. * * * A few samples have been pronounced unsafe, and a few more suspicious. Some of the unsafe samples are undoubtedly due to the length of time elapsing between their collection and analysis. To be truly representative of the water, an analysis should be made within a very few hours after the sample has been collected, and yet at least one set of samples was 5 days in reaching the laboratory—ample time for the bacteria present to develop in untold numbers.

"As for the poor samples that cannot be accounted for in this manner, you must realize that Parkersburg has practically, if not actually, the only filter plant in the State that is not using large amounts of chemicals to obtain a satisfactory water. The addition of these chemicals is a large factor in reducing the number of bacteria. If Parkersburg will slightly increase the amount of chlorine now used, the bacteria contents of her drinking water can be made as low as that in any supply in the State. I have instructed one of my assistants to tabulate the results of analyses, and I am enclosing these data herewith.

"You will note that pump house samples have but seldom shown *coli,* one of these times, 8/10/16, evidently being a fluke, as the *colon* was found, yet no bacteria were present; a second was found in a set of samples that was five days in transit, and the third and only other time in a set of samples in which both the tap and reservoir showed up remarkably well."

In consideration of such a showing as to quality and cost, and the further fact that Mr. Gray, who passed on the final design of the infiltration system in use, has a professional experience of such wide extent as to be equalled by very few of his professional brethren, it appears that Mr. Fuertes's strictures on page 631* might have been best withheld until the Parkersburg system is more thoroughly tried out.

The writer omitted descriptions of the filters and their operations for the reason that they could be given better by Mr. Gray or his assistant, Mr. Leland. As they have not done this, the writer will answer briefly some of the queries suggested in reference to the general design.

* *Proceedings,* Am. Soc. C. E., April, 1917.

Mr. Hall.

TABLE 8.—TEST OF QUALITY OF PARKERSBURG WATER SUPPLY.

Date.	River.			Pump House.			Reservoir.			Tap.			Interpretation is for tap samples only.
	Total count.	*B. coli*: 1 cc.	*B. coli*: 10 cc.	Total count.	*B. coli*: 1 cc.	*B. coli*: 10 cc.	Total count.	*B. coli*: 1 cc.	*B. coli*: 10 cc.	Total count.	*B. coli*: 1 cc.	*B. coli*: 10 cc.	
JULY–DECEMBER, 1916.													
7/ 7/16	......	......	......	8	0	+	0	0	0	......	......	......	......
7/14/16	No samples of river water taken during this period.			......	......	......	160	0	+	12	0	+	?
										8	0	+	?
7/31/16				12 000	+	+	4 000	+	+	16 000	+	+	U*
8/10/16				0	0	+	20	0	+	15	0	+	?
9/ 2/16				......	......	......	1 200	0	+	1 500	0	+	?
										950	0	+	?
10/ 7/16				300	0	0	600	0	0	3 000	0	+	?
11/ 3/16				400	0	0	400	0	0	120	0	0	S
12/ 1/16				0	0	0	0	0	0	0	0	0	S
JANUARY–JULY, 1917.													
1/ 5/17	800	+	+	50	0	+	12	0	0	30	0	0	S
1/31/17	150	+	+	3	0	0	4	0	0	7	0	0	S
3/ 7/17	Bottle broken.			15	0	0	3	0	0	Bottle broken.			‡
4/ 2/17	2 000	+	+	1 500	0	0	2 000	0	0	1 000	0	+	?
4/28/17	400	+	+	2	0	0	6	0	0	5	0	0	S
5/31/17	500	+	+	2	0	0	250	0	0	3	0	0	S
6/30/17	2 500	+	+	2 500	0	0	4	0	0	1-2 500	0	0	S
										2-1 200	+	+	U†

Explanation: + Signifies presence of *coli*. 0 Signifies absence of *coli*. * 5 days in shipment. Results of doubtful value. † Tap probably safe. No. 2 may have been accidentally contaminated in collection of sample. U Unsafe. ? Suspicious. S Safe. ‡ Tap undoubtedly safe.

Mr. Hall. Each of the five units in the filter system is connected independently with the pumps and flushing tank by an 18-in. pipe, with valves for shutting off one or more units from the remainder of the system, for back-flushing or cleaning the filters. At first, 12-in. pipes were proposed, but 18-in. pipes were adopted, in order to reduce friction and vacuum, and to regulate the supply better when flushing.

The design first considered for the pump, well, and pipes provided that the strainer pipes should flow into the well by gravity, which required a well some 20 ft. deeper than built. The writer still questions the advisability of the change in the plan. The principal objection to the arrangement as constructed is the vacuum produced in the suction pipes and the resulting injury to the filter beds by the great increase in the draft on them.

In flushing through the 18-in. pipes, the flushing water can be delivered to the strainer pipes with any desired rapidity or pressure. It is true that the water issuing from the strainers will take the line of least resistance in rising to the surface, and it is almost surprising that so few blow-holes have been found. However, the writer realizes that they might become a serious source of contamination.

The best method of removing the sand and gravel covering the strainer pipes, the extent of such needed renewals, the necessity of repairing the pipes, and other requirements of operation which will occur as the system grows older, are yet to be developed. However, it is now apparent that extensive work in sand renewal or cleaning will only be required at intervals of several years. When such examination is made, should it be found that the polluted sand and gravel extends all the way down to the strainer pipes and that a coffer-dam is required for such renewal, even in that case, the cost should hardly equal half the cost of first construction. Should it be found that it is necessary to renew only the top sand, and that the work can be done in the wet, without a coffer for unwatering, it will cost only a small fraction of the first cost. Since the filters were constructed, the Government has built a dam in the Ohio River below them, which at pool stage puts about 11 ft. of water on them.

The writer is of the opinion that the City should be advised at all times of the extent and depth of the pollution, both in order to maintain the water at a higher standard of quality and also to be able to renew the affected sand and gravel before the pollution approaches so close to the strainer pipes as to affect seriously the quality of the water or make a coffer for cleaning imperative. However, thus far, the system has been operated without any such knowledge or any attempt to obtain it. In fact, except for the back-flushing and the 3 or 4 days' work done by Mr. Leland in the fall of 1916, no work

has been done on the filters since their construction, not even to the extent of taking soundings in order to discover blow-holes. Mr. Hall.

From Mr. Tolman's commendation of this system, as previously cited, it is not clear that the State Board of Health would take such action as suggested by Mr. Knowles. In fact, it appears that any city having similar favorable conditions for such a water supply would have no serious trouble in obtaining the State Board's approval for a similar plant.

It may also be well to remember that the State Board is a creature of the State and the people, and this is a day when the autocrat of state, religion, and science seems to be passing.

When this system was adopted, the writer thought the conditions for a well system were so favorable that it was unwise for the City to take the hazard involved in the experimental system adopted; but his associates were so much in favor of doing so, on Mr. Gray's recommendation, that the writer finally concurred with them. The fact that the system in use has its source in the same underground supply as the proposed wells would appear to be more of a commendation than otherwise. After further study, he thinks that Mr. Knowles will conclude that there is a great difference between it and the old failing crib system tried by so many Ohio Valley towns.

On page 632* Mr. Fuertes makes a comparison of the cost of the mechanical filter system proposed by himself and Mr. Fuller, with a statement of cost of the new construction as actually made. The details of their estimate, as taken from their report, is as follows:

Intake and intake well	$4 000
Filtration plant complete	50 800
Low-lift pumping equipment	3 000
Main pumping station, including all machinery except low-lift pumps	51 000
Dike and grading	10 000
Force main to reservoir	25 500
	$144 300
Add 15% for contingencies and supervision	21 645
	$165 945

A comparison of this estimate with the statement of the completed work on pages 58 and 59,† including many things not in their estimate, is hardly relevant.

* *Proceedings*, Am. Soc. C. E., April, 1917.

† *Proceedings*, Am. Soc. C. E., January, 1917.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

DISCUSSION ON FINAL REPORT OF THE SPECIAL COMMITTEE ON CONCRETE AND REINFORCED CONCRETE*

By A. H. Rhett, Assoc. M. Am. Soc. C. E.†

A. H. Rhett,‡ Assoc. M. Am. Soc. C. E. (by letter).§—The Appendix to the report repeats the usual or standard textbook formulas for proportioning concrete beams. It has always seemed to the writer that though these formulas are normal for the analysis of a beam already built, they certainly operate in a reverse and roundabout manner for proportioning a new beam. Mr. Rhett.

One must first find p, a proportion of an unknown area, then K, a proportion of an unknown depth, then j, which is a fraction of the real depth, and then d; and, from d, one then works back to the absolute value of p.

It is perfectly possible to solve a reinforced concrete beam as directly as a steel beam, and from the same basic elements, that is, the bending moment and the assumed fiber stresses and moduli of elasticity. The writer has used such formulas for years. The formulas given in the report are derived fundamentally from the summation of the horizontal stresses in the concrete and steel; those used by the writer are derived from the moments of these forces. For instance, taking the first condition, a beam reinforced on the tension side only, and using the notation adopted, except that absolute values will be

* Continued from August, 1917, *Proceedings*.

† Previous discussion by Mr. Rhett appeared in *Proceedings* for February, 1917, p. 333.

‡ New York City.

§ Received by the Secretary, August 3d, 1917.

Mr. Rhett. considered instead of proportional ones; that is, Kd will be represented as y, and pbd as a, and, referring to Fig. 17, we would have:

$$\Sigma \text{ of moments of horizontal stresses} = \frac{1}{3} f_c\, y^2 + a\, f_s\, y_2 = 0$$

$$\Sigma \text{ of horizontal stresses} = \frac{1}{2} y\, f_c - a\, f_s = 0$$

and
$$y_2 = \frac{f_s}{n\, f_c} y,$$

from which would result, if $\frac{f_s}{f_c}$ is expressed as r, and M is the moment per unit of width of beam, say, inch-pounds per inch of width.

$$y = \sqrt{\frac{M}{f_c\left(\frac{1}{3} + \frac{1}{2}\frac{r}{n}\right)}};\ d = y\left(1 + \frac{r}{n}\right);\ a = \frac{y}{2\,r}$$

for a set of specified values such as $n = 15$, $f_s = 20\ 000$ lb., $f_c = 600$ lb., $r = 33.3$,

$$y = \sqrt{\frac{M}{864}},\ d = 3.2\ y, \text{ and } a = \frac{y}{67}.$$

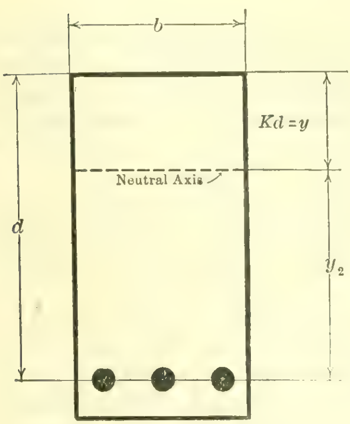

FIG. 17.

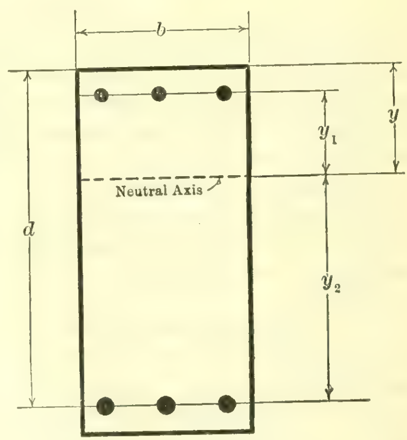

FIG. 18.

For beams reinforced on both sides, the formulas are not so easily derived, but become equally simple of application for specified values. Two assumptions would have to be made, however: first, the usual one of the ratio of y_1 to y (referring to Fig. 18); and, second, the relative area of a', the area of the steel in compression, to a. The latter assumption would be dictated largely by practical conditions, as in a series of spans of restrained beams. Let $a' = Ca$.

Mr. Rhett.

Then,

$$\Sigma M = \frac{1}{3} y^2 f_c + C a f_s' y_1 + y_2 a f_s$$

$$\Sigma H = a f_s - C a f_s' - \frac{1}{2} y$$

which give,

$$M = f_c y^2 \left[\frac{1}{3} + \frac{r}{2n} \left(1 + \frac{C n y_1}{r y} \right) \right]$$

$$y = \sqrt{\frac{M}{f_c \left[\frac{1}{3} + \frac{r}{2n} + \frac{C}{2} \frac{y_1}{y} \right]}}$$

$$d = y \left(1 + \frac{r}{n} \right)$$

$$a = \frac{y}{2r} \left(1 - \frac{C n y_1}{r y} \right)$$

$$a' = C a,$$

which equations become identical with those for single reinforcement when $C = 0$. With specified values, these become equally simple. Using the same notation and with $C = 0.5$ and $\frac{y_1}{y} = 0.75$:

$$y = \sqrt{\frac{M}{973}}$$

$$d = 3.22 y$$

$$a = \frac{y}{55}$$

$$a' = \frac{y}{27.5}$$

For T-beams reinforced for tension in stem only, and the neutral axis lying within the flange, the forms are the same as for rectangular beams reinforced on one side only, except that M and a represent values per unit width of flange and not of stem.

In the case of the T-beam the axis of which lies in the stem, very simple formulas can be similarly derived with two assumptions which are entirely within the realm of rationality, considering the very radical assumption in T-beam design in regard to the effective width of the flange. These assumptions are, referring to Fig. 19, that the averages, $f_c = \frac{1}{2} f_c$ and $Z = \frac{1}{3} t$, are both on the side of safety. Expressing t as Ky, then,

$$\Sigma M = \frac{1}{2} K f_c y \left(y - \frac{1}{3} K y \right) + a f_c y_2$$

$$\Sigma H = \frac{1}{2} K y f_c - a f_s$$

Mr. Rhett.

and

$$y = \sqrt{\frac{M}{\frac{1}{2} K f_c \left(1 - \frac{1}{3} K + \frac{r}{n}\right)}}$$

$$d = y \left(1 + \frac{r}{n}\right)$$

$$a = \frac{y}{2 r}$$

$$t = K y$$

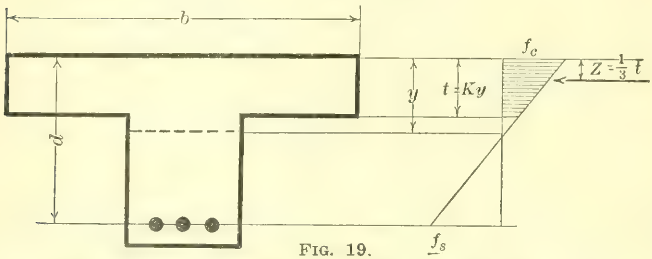

FIG. 19.

With the same specified unit values as before, and with $K = 0.75$, the formulas reduce to the same simple forms:

$$y = \sqrt{\frac{M}{675}}$$

$$d = 3.22 \, y$$

$$a = \frac{y}{67}$$

$$t = 0.75 \, y$$

All these formulas can be read directly from a slide-rule, and the actual area and spacing of the bars could be taken from a table such as Table 4, in which the moment is expressed in inch-pounds per inch of width of beam.

TABLE 4.—VALUES OF *a* FOR ROUND BARS.

DIAMETER, IN INCHES.		3/8	7/16	1/2	9/16	5/8	11/16	3/4	13/16	7/8
Area, *a*, in square inches.		0.11	0.15	0.196	0.248	0.307	0.371	0.442	0.518	0.601
Values of *a* spacing.	3 in.	0.037	0.050	0.065	0.082	0.102	0.124	0.147	0.173	0.200
	3½ in.	0.032	0.043	0.056	0.071	0.088	0.106	0.125	0.148	0.172
	4 in.	0.027	0.037	0.049	0.062	0.077	0.093	0.110	0.130	0.150

MEMOIRS OF DECEASED MEMBERS

NOTE.—Memoirs will be reproduced in the volumes of *Transactions*. Any information which will amplify the records as here printed, or correct any errors, should be forwarded to the Secretary prior to the final publication.

WILLIAM HARRY ARNOLD, M. Am. Soc. C. E.*

DIED AUGUST 13TH, 1917.

William Harry Arnold was born at Smithfield, Pa., on September 22d, 1864. From 1880 to 1883, he was a student at the Mt. Pleasant Classical and Scientific Institute, at Mt. Pleasant, Pa., and from January, 1883, to August, 1885, he served as an Apprentice in the Erecting Shops of the Pennsylvania Railroad at Pittsburgh, Pa.

In September, 1885, Mr. Arnold entered Rensselaer Polytechnic Institute, at Troy, N. Y., from which he was graduated in June, 1890, with the degree of C. E., having lost one year (1886), on account of sickness.

In June, 1890, he entered the employ of Nier and Hartford, of Chattanooga, Tenn., being engaged on general engineering work. He remained in this position and with Hartford and Hebert, until June, 1891, when he was engaged by G. W. G. Ferris and Company, of Pittsburgh, Pa., on general inspection work.

From April to September, 1895, Mr. Arnold served with the Barber Asphalt Company, of Buffalo, N. Y., and then came to New York City, as Office Manager for R. W. Hildreth and Company. He also had charge of the field work for this Company in the vicinity of New York.

From August, 1896, to April, 1899, Mr. Arnold was engaged with various officers of the Corps of Engineers, U. S. A., generally around New York City, first as Draftsman, and then as Inspector of harbor improvements. This work included the Patchogue Jetty improvement, the dredging of the South Amboy, N. J., Channel, the dredging of the Bay Ridge and Red Hook Channels, New York Harbor, as well as work in connection with battery emplacements at Fort Wadsworth, N. Y., and Fort Taylor, at Key West, Fla.

In April, 1899, Mr. Arnold entered the employ of the W. H. Beard Dredging Company, of New York City, as Superintendent and Engineer, in complete charge of the dredging fleet and the reconstruction of dredges and tugs, retaining this position until July, 1903, when he became Superintendent of the fleet of the Hughes Brothers and Bangs Dredging Company, of New York City.

* Memoir prepared by the Secretary from information furnished by E. B. Ashby, Cons. Engr., Lehigh Valley Railroad Company.

From July to October, 1904, Mr. Arnold was employed by J. G. White and Company, of New York City, on harbor work in the Philippine Islands.

In March, 1905, he was engaged by the Bush Terminal Company, of New York City, first as Superintendent of Floating Equipment, including the design and construction of tugs, car floats, dredges, etc., until June, 1906, when he was made Chief Engineer of the Company.

In 1907, Mr. Arnold resigned his position with the Bush Terminal Company to enter into private practice as a member of the firm of Arnold and Andrew, Consulting Engineers, but, in 1909, the partnership was dissolved, and Mr. Arnold became General Manager of the Bay State Dredging Company, of Boston, Mass.

From 1912 until January, 1914, he was engaged in general engineering work. On the latter date he was appointed Terminal Engineer of the Lehigh Valley Railroad Company, in New York City, which position he retained until his death, on August 13th, 1917. In this capacity, Mr. Arnold had charge of the construction of the new ore dock at Constable Hook, N. J., as well as the piers, pier sheds, and dredging in New York Harbor and at Perth Amboy, N. J.

Mr. Arnold was elected a Member of the American Society of Civil Engineers on May 1st, 1907.

JAMES EDGAR JENKINS, M. Am. Soc. C. E.*

Died July 5th, 1917.

James Edgar Jenkins was born in Vernon, N. Y., on November 25th, 1878. He received his early education in the Vernon schools, and was graduated from the village High School in 1896.

His father, Josiah W. Jenkins, is a well-known surveyor and hydraulic engineer, and from him the boy learned the rudiments of engineering so well that he qualified as Rodman with the United States Deep Waterways Survey, on which work he was engaged during 1897-98.

In 1898, the Utica Gas and Electric Company undertook the hydro-electric development of West Canada Creek, at Trenton Falls, N. Y., and, for nearly three years, Mr. Jenkins was Instrumentman and Inspector on this project.

In the mid-term of 1900, he entered Rensselaer Polytechnic Institute where he quickly overtook his class, and was graduated in excellent standing with the degree of C. E. in 1904. During his summer vacations he was employed as Instrumentman on the Rutland Canadian Railroad and the Fort Plain, N. Y., Water-Works.

* Memoir prepared by H. R. Beebe, Assoc. M. Am. Soc. C. E.

Immediately after his graduation, Mr. Jenkins entered the employ of the engineers in charge of the Pennsylvania Railroad Tunnels, in New York City, as Rodman, and shortly became Chief of Alignment on the East River Division. This led to a position as Engineer with the Naughton Company and Arthur McMullen, on Sections A and B.

For the next three years he was Engineer for the Thomas Crimmins Contracting Company, on Barge Canal Contract No. 9 and on several minor contracts in New York City.

Mr. Jenkins was then employed as Engineer for Smith, Hauser, Locher and Company, on Aqueduct Contract No. 66, which position he retained for nearly three years. While engaged on this work his health began to fail, due to pulmonary trouble, and he sought to regain it by a stay at Southern Pines, N. C. This sojourn was shortened, in order that he might undertake the alterations of the dam, spillways, etc., of the Union Manufacturing and Power Company, on Broad River, South Carolina.

Shortly thereafter, in March, 1915, Mr. Jenkins became Superintendent for the Cook Construction Company, on Contract No. 2 of the Montreal Aqueduct Enlargement. When this work was closed down by war conditions, he returned to New York and, after a brief rest, took charge of the erection of a large concrete chemical mill at Buffalo for the contractors, the John W. Cowper Company. This contract was carried out during the winter of 1916-17, and on the completion of the work Mr. Jenkins' health was so precarious that he returned to his home in New York to recuperate. Shortly afterward, however, he became associated with the writer in constructing a roundhouse at North White Plains, but, before its completion, he was obliged to leave for a sanitarium at Saranac Lake, N. Y., at which place he died of tuberculosis, on July 5th, 1917.

From the foregoing summary of his activities, it will be noted that Mr. Jenkins was never idle. In his spare moments, he was a frequent visitor to the Society House, and had a large acquaintance among the members. He was of an inventive turn of mind, and had patented several devices along mechanical lines, including an automatic flashboard and a snow-melting furnace. This furnace he had intended to try out on a commercial scale during the coming winter, and it was expected that it would be so efficient that large numbers would be used in competition with the ordinary methods of snow removal.

Mr. Jenkins' early death was due in large part to his unwillingness to recognize the signs of failing health. He seemed to have a conviction that his body was a machine which, with minor repairs, could be run indefinitely, and that overhaul could be put off safely until his complete convenience. In spite of ill-health, his mind and judgment were not affected, and his cheerfulness was the admiration of all who

knew him. That his career was cut so short is a source of deep grief to his friends and to those acquaintances who recognized in his professional attainments the promise of a bright future.

In June, 1907, Mr. Jenkins was married to Miss Helen Day, of New York City. He is survived by his widow and two children, a girl of seven and a boy of five, and also by his parents, Mr. and Mrs. J. W. Jenkins, of Vernon, N. Y., and one sister, Mrs. Carl H. Dudley, of Silver Creek, N. Y.

Mr. Jenkins was elected an Associate Member of the American Society of Civil Engineers on December 5th, 1906, and a Member on March 14th, 1916.

EUGENE CASTNER LEWIS, M. Am. Soc. C. E.*

Died February 13th, 1917.

On the left bank of the Cumberland River, about eight miles above Dover, the county seat of Stewart County, Tennessee, there may still be seen the ruins of the once famous Cumberland Iron Works, which were destroyed by the Federal Army after the capture of Fort Donelson, on February 16th, 1862.

In 1830, two brothers, John and Samuel Stacker, came from Pennsylvania to Tennessee, and in beginning this enterprise became the founders of the iron industry in this section of the State. With them came George T. Lewis, then a lad sixteen years of age. Thirty-two years later, this property had developed into a great domain of more than 60 000 acres, from which were obtained the ore, limestone, and charcoal used in its furnaces. The Hon. John Bell, James Woods, and the Yeatmans had become interested, and George T. Lewis was General Manager of the works and the affairs of the partnership. The chief product of the foundry was the massive iron kettles used in the manufacture of sugar in the Southern States and abroad. The furnaces supplied the charcoal, pig iron, and blooms used in the foundry and rolling mill, the surplus being sold on the market. The product of the rolling mill was bar iron of exceptionally high quality. About 500 slaves and many employed men supplied the labor.

Mr. George Thomas Lewis, the efficient Manager of this great property, and his wife, who was Miss Margaretta Barnes, of Philadelphia, Pa., were the parents of six children, all born at their home at the Cumberland Iron Works. The fourth child, born June 22d, 1845, was Eugene Castner Lewis, the subject of this memoir.

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* Memoir prepared by the following Committee: Messrs. Wilbur F. Foster, Hunter McDonald, John Howe Peyton, and Richard Montfort, Members, Am. Soc. C. E.

Born and reared in the very midst of organized and productive labor, the young lad inhaled with the air that he breathed the instinct and inspiration that abided with him through all the stirring years of his life. His motives and purposes were ever for production and construction; never for the reverse.

At his father's home, and at a private school in the neighborhood, he learned the rudiments of education. This was followed by a course at Stewart College, in Clarksville. Then came the four years of war between the States. Fort Donelson, only nine miles from his home, was the scene of fierce fighting and final capture. The gunboats came up the river, and his father, bound and taken as a prisoner on one of them, was forced to face and witness the destruction of the plant, the work of his lifetime. Thrilled with anger and indignation, the lad, Eugene, was so unguarded in his talk, so eager to help the Confederates, that he was threatened with imprisonment. This, however, was waived on condition that he be sent out of the State; and thus it was that Eugene became a student at the Pennsylvania Military Academy, then at West Chester, Pa., from which he was graduated with the Class of 1865 as one of ten, most proficient in mathematics and kindred studies. Here, also, he acquired a title, due perhaps to his proficiency, popularity, or characteristic leadership, a title that stayed with him to the end of his days, and for more than fifty years he was only and always known as "Major Lewis." No other title seemed to fit.

With very brief delay after his graduation from the Military Academy, Major Lewis began his professional career, in October, 1865, in the service of the Memphis, Clarksville, and Louisville Railroad, as Assistant Engineer, with the late William D. Pickett, Hon. M. Am. Soc. C. E.

In May, 1866, he entered the service of the Louisville and Nashville Railroad, as Assistant with the late Rudolph Fink, M. Am. Soc. C. E., in the Road and Engineering Department, and, from October, 1869, until November, 1870, he was in charge of the construction of the Glasgow Branch of that road.

He was then engaged in making surveys for three projected lines of railroad in Western Tennessee and Northern Mississippi until July, 1871, when he was placed in charge of the construction of the road which is now the Owensboro and Russellville Division of the Louisville and Nashville Railroad.

On completion of this service, Major Lewis withdrew from strictly professional engagements as a Civil Engineer; and, yet, in the multifarious affairs of his active life, his education for the Profession, together with his native talent, experience, and close study of engineering work, constituted a most important factor in his equipment for the conception, organization, and successful accomplishment of

the many notable achievements in which his skill and training as a Civil Engineer were in constant service.

As the business member of the firm of W. B. Read and Company, Major Lewis was engaged, in 1873-74, in the execution of a contract for the construction of certain sections of very heavy work in Kentucky, on the railroad which is now a part of the Queen and Crescent System.

Through the large purchase of explosives used on this work, Major Lewis became favorably known to the Dupont Powder Company. This led to the acquaintance and personal friendship of its owners, and to his appointment as their agent for the sale of the product of their subsidiary mills at Sycamore, in Cheatham County, Tennessee, and, later, to his entire control and management of that property, until, in 1904, manufacture at that plant ceased, and the mills were dismantled.

Sycamore Mills was the business headquarters and residence of Major Lewis from 1875 to 1897, and, during the summer, until 1907. The memory of his public spirit and usefulness is cherished and honored in that community, where the many improvements in highways, bridges, and other details still bear testimony to his skill, energy, and good judgment. It was in the reconstruction of one of these bridges that the accident occurred which finally cost him his life.

It was during the period of his engagement in this enterprise that Major Lewis was married, on October 12th, 1880, to Miss Pauline Dunn, of Nashville, Tenn., and, at their home at Sycamore, seven children were born, all of whom are living. Mrs. Lewis died at Nashville, on January 14th, 1902.

In the executive department of railways, Major Lewis served with great ability and success: On December 8th, 1896, he was elected a member of the Board of Directors by the Stockholders of the Nashville, Chattanooga, and St. Louis Railway, and served faithfully and efficiently in that capacity until his death. He was Chairman of the Executive Committee from December 14th, 1898, to October 13th, 1914, and was "Chairman of the Board" from February 28th, 1906, until his death. Twice the duties of Acting President were added to his other service; first, from February 17th to February 28th, 1906; and, again, from December 19th, 1913, to April 1st, 1914.

For more than 30 years, Major Lewis possessed the friendship and confidence of the chief officials of the Louisville and Nashville Railroad System, and by them was entrusted with the care of its interests in many matters of importance, especially in its affairs, and the development of its facilities in Tennessee. An important item in this service was the following:

"The Louisville and Nashville Terminal Company" was chartered on March 22d, and organized on March 28th, 1893, for the purpose of perfecting joint terminal facilities for the use of the Nashville, Chat-

tanooga, and St. Louis Railway, and the Louisville and Nashville Railroad, and the erection of the necessary buildings required for the joint or separate use of those roads.

The steps preliminary to the purposes of this charter were all under the personal direction of Major Lewis, and extended over a period of several years, required for the purchase of the necessary real estate. Major Lewis was made President of the Company, on March 2d, 1898, and served in that office until November 27th, 1905. In that capacity, he had responsible control of every detail, including the approval of all plans and designs, and the awarding of contracts; also, the general supervision of grading the grounds, construction of the Union Passenger Station and other extensive buildings, steel viaducts, bridges, yard trackage, and other details, altogether involving an expenditure of more than $2 500 000. This work was begun in August, 1898, and formally opened for service in October, 1900.

The Nashville and Decatur Railroad, beginning at Nashville, Tenn., and ending at Decatur, Ala., is under lease for a long term of years to the Louisville and Nashville Railroad, but the organization of the Lessor Company is maintained, and, of this, Major Lewis was a member of the Board of Directors, and Vice-President at the time of his death.

The Napier Iron Works are in Lewis County, Tennessee, and, from the time of its organization in 1890, until 1895, Major Lewis was President of the Company and General Manager of the works. A branch line of the Louisville and Nashville Railroad was built from Summertown to Napier, and the furnace plant at that place was built and operated as a charcoal furnace until 1895 under his management.

By the Courts, Major Lewis was entrusted with the following responsible duties:

In 1901, he was made Co-Receiver in the affairs of the Nashville Street Railways. This resulted in the re-organization of the Company under the title of The Nashville Railway and Light Company, in 1903.

In June, 1910, he was appointed Co-Receiver in the affairs of the Bon Air Coal and Iron Company. In the performance of this trust, he continued until his death.

It is probable that his notable success in the organization, development, and management, as Director General, of the Tennessee Centennial Exposition, held at Nashville in 1897, is the especial achievement by which Major Lewis was most widely known, and for which he justly received the greatest commendation from the public.

It is impossible, in this memoir, to give adequate or even partial description of this memorable and historic event and its far-reaching influence for good. Suffice it to say that it was the unanimous and unqualified action of the large and very able Board of Directors, as well as the voice of the people at large, that the complete success of

the Exposition was, in great measure, due to the skill, inventive genius, and admirable taste of Major Lewis in the design and construction of details, and to his rare administrative ability in the management of its affairs.

As a natural consequence, and under the inspiration of the example set in the attractive beauties of the Exposition grounds, there came the creation of the Park System of the City of Nashville; and from 1901 to 1913, Major Lewis was the moving spirit, the presiding genius of the Board of Park Commissioners. Evidence of his taste, skill, and originality is seen in numberless details, sometimes unique, always attractive, in the many Parks of Nashville, to which he devoted much time and study. At the Centennial, "The Parthenon", an accurate reproduction of the original at Athens, still stands as a memento of the Exposition.

The Nashville American had long been the leading morning daily newspaper of the city. In 1896, Major Lewis became its owner and publisher, and continued as such until 1909. Under his management, it was a vigorous and influential advocate of the policies of the political party of which it was the organ, and of integrity and "up-to-date" methods in municipal affairs. One especial reason for its purchase in 1896 was for its service in the Publicity Department of the Centennial Exposition.

The Engineering Association of the South was organized at Nashville, on September 18th, 1889, and Major Lewis was one of the charter members. He served as Director for several terms, and was President of the Association in 1893.

The following are some of the personal attributes of Major Lewis, which combined in the formation of a character, aggressive, public spirited, and of conspicuous usefulness in public affairs and in private life: Strict integrity and faithfulness in official station, and in matters of public or private trust; and, in this, he was imperative in demanding the same from colleagues and subordinates. Fearless disregard of public opinion or criticism, in advocating or executing measures which, in his judgment, were commendable and right; independence of thought, and great originality in conception, design, and methods; untiring energy and industry. His active mind, dominating a somewhat frail physique, seemed never to be at rest.

At his home, there was ever a rare and genial hospitality and an unfailing and active interest in the welfare of the people among whom he lived.

Major Lewis was gifted with correct taste and appreciation of works of art, and was a liberal subscriber to funds for the promotion thereof, as well as to other measures of public character. His private charities are known to have been large, but were ever carefully concealed.

Briefly, Major Lewis was public spirited, self-reliant, masterful, fearless, outspoken, faithful in duty, talented, successful, charitable, and honorable in all his dealings.

Like all men of strong character, he was not faultless, nor without vigorous opponents and some personal enemies; and, yet, by one and all, his death was felt to be a calamity; and his imperative dictation, that there should be "no flowers, no funeral, no fuss" when he should pass away, could not prevent the large attendance of eminent men, long-time associates, and warm-hearted friends, from near and far, at the brief but deeply impressive service, when, at the twilight hour, his remains were placed in the family vault at Mt. Olivet.

Then, too, as the twilight faded, for 300 sec. all traffic ceased, no wheels were in motion, on the entire system of the Nashville, Chattanooga, and St. Louis Railway, and on the street railways of the City of Nashville.

Major Lewis was elected a Member of the American Society of Civil Engineers, on March 5th, 1873, and served as a Director in 1903-05 and 1912-14. He valued his membership in the Society very highly, and, until his death, maintained an active interest in its affairs.

ALEXANDER WILLIAM MacCALLUM, M. Am. Soc. C. E.*

Died February 3d, 1917.

Alexander William MacCallum was born in Edinburgh, Scotland, on July 25th, 1859.

He served an apprenticeship of four years to a mechanical trade at the shops of the Central Railroad of New Jersey, at Ashley, Pa. This apprenticeship was finished in 1881, and he then entered Lafayette College, at Easton, Pa., from which he was graduated in June, 1885, with the degrees of C. E. and M. E.

During 1886, and a part of 1887, Mr. MacCallum was employed as Engineer and Assistant Superintendent at the Massillon Engine Works.

In 1887 he was engaged by Samuel R. Bullock and Company, of New York City, as Engineer and Superintendent of the water-works at Massillon, Circleville, and Defiance, Ohio, thus entering the field of water-works engineering, in which he afterward specialized.

In March, 1889, Mr. MacCallum was appointed Engineer and General Manager in charge of all the public utilities of R. D. Wood and Company. While in this position, he supervised the building and rebuilding of many of that Company's water-works plants, as well as

* Memoir prepared by the Secretary from information furnished by George W. Fuller, M. Am. Soc. C. E., Robert C. Wheeler, Assoc. M. Am. Soc. C. E., and Harry S. Hopper, Esq.

the management of several of its gas and light plants, including those at Denison, Tex.; Mobile, Florence, and Tuscaloosa, Ala.; Pensacola, Tampa, and Lake City, Fla.; Macon, Ga.; Vincennes and Greencastle, Ind.; Sharon, Corry, Chester, and Marcus Hook, Pa.; Owego, N. Y.; and Millville, N. J.

On numerous occasions, he acted as Appraiser and Arbitrator for various companies, notably for the Trinidad, Colo., Water-Works (1896); the Macon, Ga., Gas Light and Water Company (1910); the Warren, Pa., Water-Works (1911); the Ashland, Ky., Water Company (1911); as well as the Kane, Pa., Water Company, the Catlettsburg, Kenova, and Cereda, W. Va., Water-Works, etc.

From March, 1906, until his death, Mr. MacCallum was Consulting Engineer and General Manager, in charge of a number of public utilities for financial interests, with headquarters at Philadelphia, Pa., devoting his spare time to valuations and appraisals and to examinations and reports, for bankers and private investors, on water-works properties in various States. He was probably best known as the Secretary and General Manager of the New Chester Water Company, of Chester, Pa., and also of the Linwood, South Chester, Ridley, Chichester, Edgemont, and Delaware Water Companies, and of the Greencastle and Vincennes, Ind., Water Corporations.

On February 3d, 1917, while apparently in the best of health, Mr. MacCallum suffered a stroke of apoplexy which caused his death.

On October 13th, 1887, during his residence in Massillon, Ohio, Mr. MacCallum was married to Grace F. Weaver, of Cleveland, Ohio. He is survived by his wife and five children, Mrs. Nellie Williams, Jane Gordon, Grace Elizabeth, Walter Wood, and Alexander William MacCallum, Jr.

His most striking characteristic was his cheerfulness and optimism and his devotion to his work, his family, and his immediate friends, among whom his loss will be keenly felt. Although exceedingly modest and retiring, he was withal a shrewd business man. He used sound judgment in the application of his engineering knowledge in the water-works field, and won the confidence alike of citizens and public officials, as well as of his associates, in the management of the various water-works properties. Under his quiet manner, however, Mr. MacCallum had great strength of character, devotion of purpose, and deep religious convictions.

He had made his home at Merchantville, N. J., for many years, and always took an active interest in local public affairs. Among other things, he served for a long time as a member of the Sewerage Commission which inaugurated and constructed the improved sewerage system for the town.

At a meeting of the Executive Committee of the New Chester Water Company, on February 21st, 1917, Mr. Harry S. Hopper, the

Treasurer of the Committee, who was also an intimate friend of Mr. MacCallum, made the following statements:

"It is with a feeling of deep sadness that I report the sudden death on February 3d, 1917, of our late Secretary and General Manager, Mr. Alexander W. MacCallum.

"The loss of Mr. MacCallum seems irreparable to me personally, and also to the Water Works he managed, as well as the Water Works community generally. He was connected with our interests for nearly 30 years, and was much more than the title given him indicates. He took a deep personal interest in each plant and could not have been more faithful and painstaking had he owned the properties. He always put the 'ladies' first, meaning Mrs. Little, Mrs. Lippincott, and Mrs. Hopper, and never spared himself where their interests were involved.

"He built the properties up in the last ten years to a point of efficiency and profit toward which he had been aiming, and looked forward to the completion of the work planned at Chester as the crowning effort of his life; it is difficult to understand why he should not have been allowed to realize his desire."

The following resolutions were also unanimously adopted at the meeting:

"*Resolved,* That, in the death of Mr. MacCallum the Company has met a loss that cannot be supplied. Mr. MacCallum's character included an extraordinary combination of qualities. To a thorough technical knowledge of the scientific branches of the business, which he had acquired as engineer, he united the broad knowledge of business, which he had earned by conscientious devotion to work, and he had also the higher qualities which the experience of responsibility cheerfully and successfully borne brings to all. His sympathy with human nature was deep and evident, and his qualities of friendship were sincere and enduring.

"*Resolved,* That the Board desires to extend its deepest sympathy to Mrs. MacCallum and her family, and directs that a copy of these resolutions be forwarded to her."

Mr. MacCallum was elected a Member of the American Society of Civil Engineers on September 2d, 1914.

DAVID WENDEL SPENCE, M. Am. Soc. C. E.*

DIED JUNE 28TH, 1917.

On September 22d, 1868, David Wendel Spence, the son of the late Joseph and Margaretta (Wendel) Spence, was born at Austin, Tex. His early education was obtained at private schools in Austin, and, for four years, he was a member of the classes in the late Professor Bickler's Academy. In June, 1889, he received his B. Sc. degree

* Memoir prepared by J. C. Nagle, M. Am. Soc. C. E.

from the then young University of Texas, his work as a student having been in the Civil Engineering course.

During the summer of 1889, Mr. Spence was a member of the geological field party of the late Dr. Theodore B. Comstock, then Geologist in charge of the investigation of the Central Mineral District of Texas, on the Texas State Geological Survey. In September, 1889, he entered the University of Michigan where he pursued advanced work in civil engineering under the late Charles E. Greene, M. Am. Soc. C. E., then Professor of Civil Engineering in that institution, and spent one year in resident study. In 1891, after having had one year's active practice in his profession, at Denver, Colo., he received his C. E. degree from the University of Michigan.

Mr. Spence's year in Denver was spent in the office of the Lane Bridge and Iron Works, the factory of which company was in Chicago, Ill. Later, he returned to Chicago, and was Chief Draftsman of the South Halsted Street Iron Works until September, 1892, when he resigned to return to Texas to accept a position in the Agricultural and Mechanical College of Texas, with which institution he maintained continuous connection until his death. During his twenty-five years of service at the College, he occupied, successively, the following positions: 1892 to 1898, Assistant Professor of Civil Engineering and Physics, and of Drawing; 1898 to 1899, Associate Professor of Civil Engineering and Physics, and Drawing; 1899 to 1904, Professor of Physics; 1904 to 1908, Associate Professor of Civil Engineering; 1908 to 1913, Professor of Structural Engineering; and, 1913 to 1917, Dean of the School of Engineering, and Professor of Civil Engineering.

Dean Spence organized the Texas Engineering Experiment Station, at the Agricultural and Mechanical College, and was its Director from its inception until his death. He also rendered additional valuable service to the College along several lines of physical development. From 1907 to 1911, he was in charge of repairs to buildings, and of the maintenance and extension of the water supply and sewer systems of that institution. He was also Supervisor of Construction of the $100 000 mess hall, built at the College in 1913. From 1912 until the time of his death, Dean Spence was Supervising Engineer for all new construction work at the Prairie View, Tex., Normal and Industrial College (for negroes), over which the Board of Directors of the Agricultural and Mechanical College has jurisdiction. From 1911 until his death, he had complete charge of the repairs to buildings, and of water-works and sewer extensions, at Prairie View.

During his vacations, Dean Spence occupied various positions, among which the following may be mentioned: 1894, Assistant Engineer, Brazos and Burleson Railway Company; 1895 and 1896, Engineer, Brazos River Channel and Dock Company, Velasco, Tex.; 1899, Draftsman, Union Bridge Company, Athens, Pa.; 1900, Assistant

Engineer, Houston, East and West Texas Railroad Company; 1901 Draftsman, Columbia Bridge Company, Carnegie, Pa.; 1903, Assistant Locating Engineer, Gulf, West Texas and Pacific Railroad Company; and 1905, Draftsman, Penn Bridge Company, Beaver Falls, Pa.

Dean Spence also did considerable private work as Consulting Engineer, especially on highway bridges.

Among his former students are numbered many prominent engineers, not only of Texas, but of other States and of foreign countries, every one of whom bears him in affectionate remembrance, and who, together with a host of other friends, deeply mourn his untimely end. Zeal for his work, and love for his institution, caused him for years to overtax himself, and to disregard the warnings of failing health. Almost up to the very hour of his death, he was busy planning betterments in his college courses and improvements in the conduct of his Department. Long will the College miss his wise counsel and energetic efforts in its behalf.

In June, 1893, Mr. Spence was married to Miss Lucy Reese, daughter of the late distinguished Judge T. S. Reese, at Hempstead, Tex., who, with one son (who also is a civil engineer), and two daughters, survives him.

Dean Spence was elected a Member of the American Society of Civil Engineers on October 1st, 1913.

ERLE LeROY VEUVE, M. Am. Soc. C. E.*

Died March 25th, 1917.

Erle LeRoy Veuve was born at the Military Post, at Port Townsend, Wash., on January 21st, 1877. His early education was received at Post Schools in many different parts of the country.

In 1893 he went to work as Chainman for the Northern Pacific Railway Company and attained the position of Levelman. In the fall of 1895, he entered Troop Polytechnic Institute, at Pasadena, Cal., with the intention of taking up the profession of Civil Engineering. In 1896 he entered the Civil Engineering Department of Stanford University, where he continued his studies until 1900. During this time, Mr. Veuve's summer vacations were spent in engineering work on the Northern Pacific and Southern Pacific Railways, and for irrigation companies in the San Joaquin Valley, California. During a part of 1898 he served as Chief Yeoman in the United States Navy.

During the latter part of 1900 and the early part of 1901, Mr. Veuve had charge of a number of short pieces of work. In March, 1901, he became Engineer in charge, for the United Verde and Pacific Railway Company in Arizona, superintending the extensive reconstruction

* Memoir prepared by A. M. Strong, Assoc. M. Am. Soc. C. E.

on that road. On the completion of this work, in April, 1902, he was employed as Assistant Chief Engineer of the Pacific Electric Railway Company, in Los Angeles, Cal., first in charge of location and, later, of construction. Mr. Veuve handled much of the extensive development work required in the construction of the Company's inter-urban system. In addition to the railroad lines, this work covered a wide field of engineering and construction.

In 1908, further extensions of the Pacific Electric Railway having been discontinued, Mr. Veuve resigned and went into private practice as Consulting Engineer, with offices in Los Angeles. He was engaged in this practice at the time of his death, which occurred very suddenly and unexpectedly on March 25th, 1917, from cardiac asthma. During the last two years of his life, he had been associated with the writer, under the firm name of Veuve and Strong. As a Consulting Engineer, Mr. Veuve's practice covered a wide field, though, in the main, it was connected with the preliminary investigations of electric railway, hydro-electric, and irrigation projects, much of which was of a confidential character. His ability in this field was well known, and his loss will be felt by an exceptionally large circle of friends and business associates.

On October 27th, 1908, Mr. Veuve was married to Miss Anna M. Heim, of Santa Rosa, Cal. He is survived by his widow, his mother, a son, and a daughter.

Mr. Veuve was elected a Junior of the American Society of Civil Engineers on September 3d, 1901, and a Member on February 2d, 1909. He was also an Associate Member of the American Institute of Electrical Engineers, a member of the Engineers and Architects Association of Southern California, and of the Jonathan Club, of Los Angeles, Cal.

GEORGE GERE MacCRACKEN, Assoc. M. Am. Soc. C. E.*

Died August 1st, 1913.

George Gere MacCracken, the son of Henry Mitchell MacCracken (Chancellor Emeritus of New York University), and Catherine (Hubbard) MacCracken, and a grandson of the Rev. John Steele MacCracken, of Ohio, and the Rev. Thomas Swan Hubbard, of Vermont, was born on March 8th, 1878, at Toledo, Ohio.

He was educated at the Lyons Collegiate Institute, Broadway and 21st Street, New York City, and, in 1894, entered the New York University School of Applied Science, from which he was graduated in 1898 with the degree of B. S., receiving the degree of C. E., in

* Memoir prepared by George H. Pegram, President, Am. Soc. C. E., and Charles H. Snow, M. Am. Soc. C. E.

1899. He was Inman Fellow at New York University during the college year of 1898-99, and also took special courses in electricity at the Columbia University School of Mines. In 1899, he was awarded the Hoe Sanitary Engineering Prize for the best work in sanitary investigation.

In 1899, Mr. MacCracken was appointed to the Engineering Staff of the Manhattan Railway Company. He remained with that Company until 1902, when he became Assistant Engineer of the Interborough Rapid Transit Company, holding that position until 1908. His principal work with these companies was the supervision of the construction of the Elevated Railway Terminal Yards at 159th Street and the Harlem River and at Third Avenue and 179th Street. These yards cost about $500 000 and required very skillful engineering.

In 1908, he organized the Alboro Contracting Company, of which he became President, and, later, the MacCracken, Hauer, Terry Company, engaging in the general contracting business.

Mr. MacCracken was of large and commanding presence and was possessed of a genial nature, although rather exclusive in his choice of associates. His thorough preparation for engineering work, coupled with his industrious and conscientious nature, gave promise of substantial accomplishment.

He lost his life by drowning off Glen Cove, Long Island, on August 1st, 1913, having been stricken by heart failure after the exertion of bringing his launch through a severe storm on Long Island Sound. He was buried in Sleepy Hollow Cemetery, at Tarrytown, N. Y.

Mr. MacCracken was married on April 29th, 1907, to Martha J. Hall, the daughter of the late John H. Hall, who, with one son, Williston Ward MacCracken, born in 1910, survives him.

During his college course, Mr. MacCracken was a member of the Psi Upsilon Fraternity, the Red Dragon Senior Society, and the Chemical Society, and was President of the Engineering Society. Later, he was a Trustee of the American Savings Bank of New York, and a member of the Railroad Club and the Psi Upsilon Club.

Mr. MacCracken was elected a Junior of the American Society of Civil Engineers on February 5th, 1901, and an Associate Member on February 3d, 1904.

WILLIAM STUART SMITH, Assoc. M. Am. Soc. C. E.*

Died November 5th, 1916.

William Stuart Smith was born in Troy, N. Y., on April 15th, 1856. He was the eldest of three children, and was descended on both sides from Revolutionary ancestors. His grandfather, Captain

*Memoir prepared by John F. Skinner, M. Am. Soc. C. E.

Levi Smith, was a contemporary of Commodore Vanderbilt in the steamboat business, and his father, William Stuart Smith, was long connected with the New York Central Railroad. His mother was Delia Marble Newton, of Bennington, Vt.

He was educated at private schools and the College of Montreal, and after removing with his parents to Rochester, N. Y., in his seventeenth year, he attended the school of Professor George Hale.

Mr. Smith's first engineering work was with a field party during the four-tracking of the New York Central Railroad. On June 1st, 1876, he entered the office of the City Surveyor, of Rochester, and, in 1883, he was made First Assistant.

In 1886, he entered the employ of the Warren-Scharf Asphalt Paving Company, which, in 1900, became The Warren Brothers Company, of Boston, Mass. Mr. Smith was also District Manager of The Warren Chemical Company, of New York City, and carried on a large roofing business. In 1914, he purchased the Western New York rights of The Warren Chemical Company, and conducted the business under the name of The W. Stuart Smith Company. He continued his connection with The Warren Brothers Company, and, at the time of his death, was a Director and Vice-President of that Company. Mr. Smith was frequently consulted and generally recognized as an expert in paving and roofing.

About fifteen years ago he was interested in a plantation operated by The Indian River Pineapple Company.

Mr. Smith was a man of fine address and had a lovable personality. He had hosts of friends whom he entertained at his summer home, Sunny Side, at Pultneyville, N. Y., on Lake Ontario. He was a lover of fine horses, and maintained a dashing span long after the automobile had come into vogue.

He was a member of many professional, social, patriotic, and athletic organizations, a Thirty-second Degree Mason, and a communicant of Christ (Episcopal) Church, of Rochester, N. Y.

On September 4th, 1879, he was married to Minnie Pomeroy Sackett, of Rochester, who survives him. He is also survived by two brothers, Charles S. Smith, of Connecticut, and Frederic Levi Smith, of Rochester, two sons, Captain Lawrence N. and Donald Stuart Smith, two daughters, Mrs. Marion Elizabeth Stafford, and Mrs. Carol Content Gard, and four grandchildren.

Mr. Smith was elected an Associate Member of the American Society of Civil Engineers on January 8th, 1902.

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OF

CIVIL ENGINEERS

VOL. XLIII—No. 8

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(INSTITUTED 1852)

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Edited by the Secretary, under the direction of the Committee on Publications.

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MINUTES OF MEETINGS

OF THE SOCIETY

September 19th, 1917.—The meeting was called to order at 8.30 P. M.; Director Alfred D. Flinn in the chair; Chas. Warren Hunt, Secretary; and present, also, 98 members and 7 guests.

A paper by Ray W. Berdeau, Jun. Am. Soc. C. E., entitled "The Three 15-Cubic Yard Dipper-Dredges, *Gamboa, Paraiso,* and *Cascadas,* as Supplied and Used on the Panama Canal," was presented by the Secretary. The subject was discussed by Messrs. W. M. Rosewater and C. E. Fowler.

The Secretary announced the election of the following candidates, on September 11th, 1917:

As Members

Frederick William Allan, Buffalo, N. Y.
Anthony Francis Bannon, Jr., Bradford, Pa.
Chester de Baun Christie, Duluth, Minn.
Charles Calhoun Cragin, San Francisco, Cal.
Cornelius Mark Daily, St. Louis, Mo.
William Worrell Drummond, Fort H. G. Wright, N. Y.
Joseph Burton Gemberling, Philadelphia, Pa.
William Everett Hawley, Duluth, Minn.
Arthur Gunderson Hayden, Albany, N. Y.
Charles Emil Kauffmann, Atlanta, Ga.
Newton Mansfield, Pittsburgh, Pa.
Charles Ward Mullen, Bangor, Me.
Seth Perkins, St. Augustine, Fla.
Herbert James Bingham Powell, New York City
Theodore Charles Roberts, New York City
James Wallace Shikles, Chicago, Ill.
Walter Sutton Syrett, Chicago, Ill.

As Associate Members

Edwin Theodore Asplundh, Bryn Athyn, Pa.
Charles Robert Barker, Auburndale, Mass.
George Rockwell Bascom, Madison, Wis.
Charles Alton Baughman, Ames, Iowa
Paul Bayliss, Jefferson City, Mo.
Robert Stanley Beard, Kansas City, Mo.
Eduardo Beaven, City of Mexico, Mexico
Tulla Ethan Bliss, Memphis, Tenn.
Emmet Chandler Blosser, Columbus, Ohio
Herman William Bluhm, Richmond Hill, N. Y.
Charles Warren Breed, Western Springs, Ill.
Dudley Chipley, Columbus, Ga.
Lothrop Crosby, Richfield, Idaho
Nelson Cunliff, St. Louis, Mo.
Brinton Brown DeWitt, Parlin, N. J.
Ernest Drinkwater, St. Lambert, Que., Canada
Latta Vanderion Edwards, Pullman, Wash.
John Walter Fairlie, New Orleans, La.
Guy Cephas Finley, Naches, Wash.
Joseph Hamilton Fleming, Columbus, Ohio
Cecil Alfred Forter, Topeka, Kans.
Myron Elmer Fuller, Philadelphia, Pa.
Norman Will Funk, Kansas City, Mo.

Raoul Charles Rodolphe Gautier, New York City
Nelson Barnes Hunt, Olympia, Wash.
Alva Guy Husted, Cleveland, Ohio
John Lansdale, Los Angeles, Cal.
James Albro Lawrie, Duluth, Minn.
Kenneth King MacKenzie, Chicago, Ill.
Francis Regis McDonnell, Charlestown, Mass.
Frederic Berkby McKinnell, Carrollton, Mo.
Franklin R. McMillan, Minneapolis, Minn.
Robert Marsh, Jr., Champaign, Ill.
Charles Culbertson May, Seattle, Wash.
John William Moffett, New Haven, Conn.
Edward Francis Morey, Dallas, Tex.
Jerome Aaron Moss, Chicago, Ill.
Russell Alger Murdoch, Detroit, Mich.
Archibald Whitfield Nance, Pittsburgh, Pa.
John Evered Norton, San Francisco, Cal.
Allen Thatcher Paine, Oneonta, N. Y.
David Harlan Plank, Chicago, Ill.
Thomas Francis Quinn, New Orleans, La.
Herbert Dwight Raymond, Americus, Ga.
Bird LeGrand Rees, New York City
Raymond Victor Rosenbaum, Martinsburg, W. Va.
Jacob Peter Sartz, Shanghai, China.
Herbert Moore Sharp, Columbus, Ohio
Harry Woodworth Shimer, San Francisco, Cal.
Walter Grover Smith, Columbus, Ohio.
Hugh Max Sprague, Great Falls, Mont.
Markley Stevenson, Rochester, N. Y.
John William Straney, Midland, Pa.
Rufus Carroll Thaxton, Austin, Tex.
Joshua Holmes Vogel, Hanover, Ohio
Wendell Douglas Volk, Washington, D. C.
Edward Ewing Wallace, San Luis Obispo, Cal.
John Cloyd Wallace, Chicago, Ill.
Anthony William Wand, Albuquerque, N. Mex.
Earle Lytton Waterman, Lansing, Mich.
Peter Weiner, New York City
Alexander Dennis Williams, Morgantown, W. Va.
Howard Stephen Williams, Atlantic, Iowa

As Associate

Robert Alexander Chiqueno Smith, New York City

As Juniors

Leo Murry Arms, Kansas City, Mo.
Peter DeMarcus Burks, Jr., Atlanta, Ga.
Alan Gordon Cherry, Washington, D. C.
Justus Gilbert Chesley, Los Angeles, Cal.
Robert Duffus Clancy, Dallas, Tex.
Robert Augustus Cummings, Jr., Pittsburgh, Pa.
Charles Dwight Curtiss, Anderson, S. C.
Eugene Weldon Fickes, Lancaster, Pa.
Clarence Malcolm Fuller, Chicago, Ill.
Frank Berry Garrett, Dallas, Tex.
Joseph Ricker Guptill, Oakland, Cal.
Lewis Adams Holborn, Hamlet, N. C.
Richard Lewis Hyde, New York City
Nathan Bernd Jacobs, Pittsburgh, Pa.
Gerald Marcy Keith, Portland, Me.
Kenneth Winans LeFever, Little Rock, Ark.
David Arthur McClung, San Diego, Cal.
Alfred Gardner Ranney, San Antonio, Tex.
Frederick Harrison Weed, New York City

The Secretary announced the transfer of the following candidates on September 11th, 1917:

From Associate Member to Member

Western Radford Bascome, New York City
Lester Van Noy Branch, Sherburne, Mont.
William Henry Connell, Philadelphia, Pa.
Herman Francois Doeleman, Baltimore, Md.
James Nickerson Gladding, El Paso, Tex.
William Joseph Gough, San Diego, Cal.
Andrew Pearson Hoover, New York City
Wesley Winans Horner, St. Louis, Mo.
Shaler Charles Houser, University, Ala.
William Nelson Jones, Minneapolis, Minn.
Dwight B. La Du, Albany, N. Y.
Richard John Lockwood, Port St. Joe, Fla.
James Leonard Lytel, Provo, Utah
Harry Deyoe McGlashan, San Francisco, Cal.
Lewis M. Martin, Atlantic, Iowa
Robert Hall Merrill, Albany, N. Y.
Charles Henry Pierce, Boston, Mass.
Frederick Horace Tibbetts, San Francisco, Cal.
Perry Topping, St. Louis, Mo.
David Crooker Trott, Kalispell, Mont.
Nathaniel Parker Turner, Marshall, Tex.

FROM ASSOCIATE TO ASSOCIATE MEMBER

ROGER LEROY MORRISON, College Station, Tex.

FROM JUNIOR TO ASSOCIATE MEMBER

JOHN WOLFRAM BERKEFELD, Piedmont, Cal.
GEOFFREY ARTHUR CAFFALL, Brooklyn, N. Y.
HERBERT GREISS CROLL, Crafton, Pa.
LESLIE CARL FRANK, Washington, D. C.
WILLIAM CHARLES GIFFELS, Jackson, Mich.
JOHN CHRISTOPHER HASKINS, Chicago, Ill.
RAY WEBB McMULLEN, New York City.
FRANK ALWYN MARSTON, Wollaston, Mass.
CAESAR MASSEI, Roanoke, Va.
STANLEY WALLACE MOORE, Springfield, Mass.
ALVIN RUSH MURPHY, New York City
RUSSELL MOORE ROBINSON, Hudson, N. Y.
WILLIAM EDWARD RUDOLPH, Brooklyn, N. Y.
HOMER J. SHARP, Los Angeles, Cal.
EARL AUDIE SMITH, Pilcher, Ky.
ALFRED KENNETH STARKWEATHER, Passaic, N. J.
ROY STANLEY SWINTON, Ann Arbor, Mich.
HARRY TIDD, Hutchinson, Kans.
EDWARD MAYO TOLMAN, Charleston, W. Va.

Adjourned.

October 3d, 1917.—The meeting was called to order at 8.30 P. M.; Director Alfred D. Flinn in the chair; Chas. Warren Hunt, Secretary; and present, also, 132 members and 13 guests.

The minutes of the meeting of September 5th, 1917, were approved as printed in *Proceedings* for September, 1917.

A paper by William Barclay Parsons, M. Am. Soc. C. E., entitled "The Cape Cod Canal," was presented by Walter J. Douglas, M. Am. Soc. C. E., and illustrated with lantern slides. The paper was discussed by Messrs. Clemens Herschel and T. Kennard Thomson.

The Secretary announced the death of RICHARD EDWARD SPEAKMAN, of Brandon, Manitoba, Canada, elected Member, January 4th, 1910; died January 13th, 1917.

Adjourned.

SOCIETY ITEMS OF INTEREST

Discussion on Relations of Local Associations to the Parent Society, to Other Engineering Organizations, and Engineers, and to the Public

The report of the Committee, appointed by the Board of Direction, on the Relations of Local Associations of the American Society of Civil Engineers to that Society, to Other Engineering Organizations, and Engineers, and to the Public, was published in *Proceedings* for May, 1917, pages 327-330.

On May 23d, 1917, the Secretary addressed the following letter to each of the Local Associations:

"I call your attention to a Report of a Committee to the Board of Direction on the Relations of Local Associations of the American Society of Civil Engineers to that Society, to other Engineering Organizations, and Engineers, and to the Public, and suggest that this whole matter be taken up for discussion by your local Association, and that the resulting discussion be published in *Proceedings*."

Reports or discussions relative to this matter have been received from the Baltimore, Cleveland, Seattle, and Colorado Associations, and are here published for the information of the membership.

Report from the Baltimore Association

The following report was adopted at the meeting of the Baltimore Association on May 2d, 1917:

"May 2d, 1917.

"To the Baltimore Association of Members,
American Society of Civil Engineers.

"Gentlemen:—The undersigned committee, appointed by President Bush at the meeting of March 21st, 1917, to consider the question of the relation of local associations to the American Society of Civil Engineers, begs to report as follows:

"We recommend that in considering this question, the committee raised in January by the Board of Direction of the Society take action with regard to this question along the following lines:

"1.—That the formation of active local associations be encouraged and fostered by the Society. It is suggested that the natural lines along which such subdivisions of the general body of the membership of the Society could be made are those of the thirteen geographical districts into which North America is divided by the Constitution of the Society; this would not mean necessarily that there would be but one local association in each geographical district, but that such local organizations as might be formed in different parts of one district should have a more or less definite relationship with the district lines as a basis.

"2.—That the ordinary expenses of carrying on the business of local associations should be paid from the annual dues of members to the treasury of the Society. In this way, members at a distance

from New York, who get very little return for their dues, except receiving the publications of the Society, would be able to feel that some part, at least, of the money paid into New York was used for their direct benefit. It would result, also, in practically every member of the Society in any district taking an active interest in the affairs of the local association of that district.

"3.—That affiliation and co-operation with the local associations of other engineering and technical societies should be encouraged.

"4.—That a definite policy should be formulated, if possible, regarding the participation of local associations in matters of civic improvement and other public activities in their respective districts. It is suggested, further, that co-operation with other local associations in this regard would be particularly helpful and effective.

"5.—That a definite policy should be formulated by the Society by which most of the questions relating to the election of officers from the separate districts be left in the hands of the membership of these districts, instead of being decided, as at present, at the Annual Meeting in New York.

"Your Committee would respectfully recommend, further, that a copy of this report be sent to Secretary Hunt, with the request that it be brought to the attention of the Board of Direction of the Society, in answer to his letter of February 2d, 1917, in which he calls the attention of this association to the appointment of the committee consisting of Messrs. Bontecou, McDonald, and Jonah.

"Respectfully submitted,

"Wm. D. Janney,
C. J. Tilden,
W. Watters Pagon,
Committee."

From the Cleveland Association

The following discussions are by members of the Cleveland Association:

"*New Members.*—This paragraph, if literally construed, would bar an Association from taking the initiative in reporting upon a candidate for admission. The Committee could hardly have intended such a prohibition. On the first page of the Preliminary Notice of Applications are several paragraphs indicating to members their duty to submit such reports and the dependence of the Board upon such communications. The paragraph in question should be re-worded to remove this apparent restriction and to conform to the present expressed practice of the Society.

"*Local Societies.*—It is very gratifying to note the attitude of the Committee as indicated in this paragraph. The Local Associations are the ones which most affect the every-day life of every man and may have the most important influences upon his career. If engineers are to retain that influence upon public affairs to which their position entitles them, it is necessary that they should first co-operate with engineers in all branches of the profession. The best means of such co-operation is through the activities of a strong local society. The form of organization and character of membership of such societies

will differ with varying local conditions. It seems wise to allow the largest degree of independence, and not to attempt to conform all local societies to a single pattern, nor to dictate the character or qualifications of membership.

"The recent movements towards co-operation and unity of action among the National Societies is suggestive and hopeful. It seems probable that there will always be a segregation of engineers along professional lines. This segregation should be national for consistent results, and all segregation along other lines should be discouraged. It is, perhaps, too soon to launch a definite programme looking to the centralization of all engineering societies in a single representative body, and the consummation of such an object is, perhaps, not possible. However, the recent growth in numbers and membership of local societies and the evident desire for local and national co-operation and for a larger sphere of influence for engineers in public affairs, indicate that it is time to give careful thought to the questions and relations involved. The report of the committee in consideration is a timely and well-considered contribution to this subject. As a further suggestion towards constructive work, would it not be feasible to require as a prerequisite to admission to one of the national segregated societies that the candidate should hold membership in the appropriate local society? Such a requirement would form a close bond of mutual interest and support between the local and national societies and almost knit all engineers into a nation-wide fellowship.

"GEO. H. TINKER."

"MR. GEO. H. TINKER, *Secy.*,

Local Section of Am. Soc. C. E.,

Cleveland, Ohio.

DEAR SIR.—Responding to your request for comments on the report of the Committee on the Relation of Local Sections to the American Society, I would suggest that possibly the intent of the committee would be expressed more clearly in the third proposed By-Law on page 328 of the May *Proceedings,* Volume 53, No. 5, if it were made to read thus: 'The Constitution may only be amended by the affirmative (or majority) vote of two-thirds of the qualified members and the approval of the amendment by the Board of Direction,' the important changes being the addition of the words, 'affirmative (or majority),' and the elimination of the word, 'first'. It does not appear to be necessary that the approval of the Board of Direction should be given before taking the vote.

"In the fourth By-Law, referring to the presentation and discussion of professional papers, the present wording will bar the presentation of papers by others than members of the American Society. It is the practice of the Society to receive papers from persons not members, and no reason is submitted why this practice should be barred to local sections.

"Concerning new members, it is stated that local sections should investigate the qualifications of candidates for admission 'about whom the Board of Direction desires information and make an official report at the request of the Secretary of the Society.' It is pertinent to inquire whether it would be appropriate for the Local Section to investigate

the qualifications of candidates for admission without the request of the Board of Direction and to report thereon without the request of the Secretary of the Society.

"This report is the first effort coming to my attention in which the American Society has indicated an interest in the affairs of the Profession generally and outside of the Society. It was the ambition of the earlier members that the American Society should be comprehensive in its membership and policies and serve the interests of the whole profession.

"With the rapid development of engineering and the differentiations in practice, there have grown up interests of such importance that other societies of great strength and influence now compete with our own. The result is that the American Society of Civil Engineers can in no way pretend to speak for the whole profession, but there is great need for an organization which shall have this power. It is idle to think of whipping all engineers into the Society or saying that members of the profession outside of the Society shall have no voice in engineering matters and policies of political or national character. In striving for the benefits belonging to a democracy, individual engineers and small groups of engineers will demand consideration of their interests just as citizens do in matters affecting their citizenship.

"The needs of the Profession in this respect will not be met until there shall come into being a national organization to which any group of engineers can look hopefully for assistance in its own particular needs. An unlimited diversity of procedure in dealing with legislation, ethics, and public affairs in the different states and cities is chaotic and harmful and defers the time when the Profession can enjoy the advantages of being a single, compact, and orderly unit in the national commonwealth.

"It is only by the attainment and maintenance of such a position that we may hope to thrive sucessfully and hold the place to which we aspire in relation to other professions and organized groups of citizens. The best efforts of our most thoughtful leaders should be concentrated on this problem, and we should be assured that, as with other problems of this class, it must be solved either in a well-planned and efficient manner or in the slow, tedious, and painful process of gradual development. It is within our power to make a choice, and it is to be hoped that those who are in a position to act will choose wisely.

"Yours truly,

"A. J. Himes."

From the Seattle Association

The following report was adopted by the Seattle Association at its meeting of August 27th, 1917:

"The report of Messrs. Bontecou, McDonald and Jonah contemplates an increase in the number of Local Associations, an extension of their functions, and suggests that the name be changed to Local Sections.

"The Seattle Association heartily endorses the spirit of the report and the greater part of the wording as far as it goes, but it feels that sufficient responsibility has not been delegated to the Local Sections

to carry out one of the main points the Committee has in view, that of making the Society truly National in character and in influence.

"As a means of securing this, it suggests that:

"*First.*—One or more delegates from each local section be sent to the Annual Meeting, with expenses paid by the Society, and that definite responsibility in the conduct of the business of the Society be imposed upon these delegates.

"*Second.*—All applications for membership be considered by the Local Section having jurisdiction before being acted upon by the Board of Direction.

"*Third.*—Professional papers be first presented to a local section except those that may be selected for annual meetings or conventions.

"*Fourth.*—A sufficient proportion of the annual dues be remitted to the sections to care for the necessary expenses.

"In order to carry out these provisions, it would apparently be necessary to install one or more (preferably several) Local Sections within the boundaries of District No. 1. This would imply that the relations of the Local Sections in District No. 1 to the headquarters' office would be practically the same as in other districts, and that the dues to the Parent Society should be uniform in all Districts. It is therefore suggested further that:

"*Fifth.*—The work of the headquarters' office be developed along the line of general administration including the appointment and direction of standing committees, issuing the publications and arranging for annual meetings and conventions, with the added responsibility of extending the influence of the Society through Local Sections.

"*Sixth.*—Local Sections shall consist of not less than twenty-five (25) nor more than five hundred (500) members of all grades.

"AUGUST 27, 1917."

The following letter supplements the report adopted by the Seattle Association:

SEATTLE, WASH., Sept. 17, 1917.

"MR. CHAS. WARREN HUNT,
Sec'y, Am. Soc. C. E.,
220 West 57th St., New York.

"DEAR SIR.—Under date of May 23, you suggested that the Local Associations take up for discussion the Bontecou Report on the Relations of Local Associations to the Parent Society, which appeared in the *Proceedings* for May, 1917. In the several meetings which the Seattle Association has held since then it has given this matter consideration, the results of which are embodied in a brief committee report, prepared by Messrs. A. H. Fuller, E. B. Hussey, A. S. Downey, and P. A. Franklin, copy of which is herewith enclosed. The President was instructed to supplement the committee report with such general remarks upon its subject matter as seemed to him desirable, and in forwarding this material it is hoped that it will receive publication in the *Proceedings*, as suggested in your letter, to facilitate and encourage an exchange of views and opinions of the various Local Associations upon this important matter.

"In respect of the first suggestion of the Committee report, we believe that an organization with 7 000 corporate members has outgrown its swaddling clothes and that the voluntary assembly or town-meeting form of 'Business Meeting', as now provided by the Constitution, should be superseded by something less archaic and more adapted to the present needs of the Society. We believe that a delegated organization, with delegate expenses borne by the Society, is the proper successor of the present system, and that the Locals afford the best basis for delegate representation. Such an arrangement would insure a proper geographic distribution of representation at the Business Meetings, would stimulate a wider interest in Society affairs and the more serious consideration of important matters that may come up or that might be entrusted to such meetings, and it would, moreover, keep the Society in closer and more constant touch with the membership at large. This implies a radical recasting of Article VIII of the Constitution and with that recasting, among other details, should be considered whether or not the 'Business Meetings' of the delegate organization should not be limited to one a year, the minimum and maximum number of delegates to be allowed each Local and to what extent and how representation shall be based upon the numerical membership of the Local.

"In respect of the fourth suggestion of the committee report, we believe that, in order to encourage greater activity in the Locals, there should be remitted back to them a portion of the annual dues paid to the Parent Society. This is the practice of the American Institute of Electrical Engineers, the American Society of Mechanical Engineers and the American Institute of Mining Engineers, and it is our advice that it has proven an advantageous and highly satisfactory arrangement for all concerned. It would seem then that our Society should adopt a similar course, and the amount of dues to be remitted, in the opinion of this Association, should be sufficient to meet the ordinary expenses of the Local, say 20% of the annual dues paid to the Parent Society.

"The hope and inspiration of the proposals herein made is to strengthen the local associations and to see them become the active, energizing units of the Society. We believe that they can thus be made the means of healthily expanding the membership and usefulness of the Society and, among other things, of making it more capable of serving the public interest as well as the immediate interest of its membership. The scheme implies the compulsory organization of locals, all members resident within the boundaries of any Local, as established by the Society, becoming members of that Local automatically, and the remission of dues could be on the basis of that membership. District No. 1, as well as other Districts, would thus have several Locals and the relationship of all Locals to the Parent Society would be identical, with identical membership dues, etc. It would remove the undesired distinction between Resident and Non-Resident members, with their inequality in dues, as now provided in Articles III and IV of the Constitution, and provision could be made that, to the extent that the Society headquarters were given over to the use of District No. 1 Locals, a rental charge would be made therefor.

"We believe it is none too early for the Society to adopt some such line of action as herein indicated and that, in any event, the time has

arrived for a careful consideration of the matter by the Board of Directors and by the Locals already organized.

"Very respectfully,
"JOSEPH JACOBS,
"*President,* Seattle Association
of Members, Am. Soc. C. E."

Report of Discussion by the Colorado Association

The following discussion took place at the meeting of the Colorado Association on June 9th, 1917:

CHAIRMAN JAYCOX.—The Secretary will please read the communication from the Secretary of the Society.

(The Secretary read the letter from the Secretary of the Society.)

SECRETARY HINMAN.—This matter is up for discussion at the request of the Secretary of the Parent Society.

CHAIRMAN JAYCOX.—I suggest that you read an abstract of the report of the Bontecou Committee. I think the quickest method would be to discuss the subjects as they are read. Please read the first one.

SECRETARY HINMAN.—The report mentions the matter of co-operation with other organizations, and states that the plan of district representation was approved by the Conference of fourteen Associations in 1915, but at the request of the Secretary, in February, for an expression of opinion, only four Associations responded, and only one advocated the district plan. All Associations which did offer suggestions agreed that they wanted few restrictions, and freedom to deal with local affairs. The Committee recommends that the divisions be known as "Sections" of the American Society of Civil Engineers. The following is its report. The first heading is "Rules."

"*Initiation.*—A Local Section may be authorized by the Board of Direction at the written request of fifteen Corporate Members of the Society, and must consist of at least twenty-five members of all grades residing within the territory covered."

"*Constitution and By-Laws.*—The Constitution and By-Laws of a Local Section must be approved by the Board of Direction. The Constitution should state, in effect:

"*First.*—The objects to be attained are the advancement of engineering knowledge and practice; the cultivation of friendly relations with all engineers; the maintenance of high professional standards; and co-operation with other engineering societies, with a view to promoting the general welfare of the American Society of Civil Engineers and the Engineering Profession."

Do you want to report on this, Mr. Jaycox?

CHAIRMAN JAYCOX.—I don't think so. If you will start with the heading "Policy", whatever discussion may be taken up on each section will be about all that will be required of us under the direction of the Secretary.

MR. JOHN E. FIELD.—What is the idea of changing the name from "Association" to "Section"? "Association" seems to express it much better than "Section."

CHAIRMAN JAYCOX.—The expression of the Association should be taken on that.

MR. FIELD.—In nearly all conventions and meetings where the word "section" is used it rather indicates a difference in the subject matters to be taken up. There will be different sections to discuss different subjects, and this, to my mind, is an association. We are associated together for social and professional work, and I, for one, much prefer to retain the old name of "Association."

CHAIRMAN JAYCOX.—The argument of the Committee is that as the word "Association" is more properly applicable to a local organization of branches or sections of the National Societies, and because the word "Sections" more nearly suggests the close relation to the Society which is sought to be established, and besides is a more convenient word, the Committee recommended it.

MR. G. N. HOUSTON.—Up to the present, or at least until just recently, the Parent Society has not been over-anxious to recognize these branches. When we organized it was very careful not to father the thing too much, and now it is very anxious to do the other thing, which is to have them directly connected with the Parent Society, and I think that is the reason for changing the name to "Section." "Association" suggests rather a body which has been gathered together, without the sanction, perhaps, of the Parent Society, and a "Section" would be directly under the sanction and directly a branch of the Parent Society.

CHAIRMAN JAYCOX.—Any further remarks in reference to the choice of the word? Do you wish to make any motion on this question as an expression of opinion of the Association?

MR. E. A. MORITZ.—Mr. President, Mr. Houston has used the term "Branch." It seems to me that that would be the best term—Branch of the American Society of Civil Engineers. I agree with Mr. Field that the term "Section" is not good, because it designates a branch that considers certain parts of the Society work, whereas these local branches consider all fields.

CHAIRMAN JAYCOX.—In order to get this to the Parent Society, it would be advisable to have a motion on the adoption of our idea of the preferable word.

MR. FIELD.—Mr. Chairman, I move that this Association suggest to the Parent Society that the word "Association" be used.

MR. HOUSTON.—I second the motion.

CHAIRMAN JAYCOX.—Gentlemen, you have heard the motion made by Mr. Field and seconded by Mr. Houston. Any remarks?

MR. D. F. BLACK.—Mr. Chairman, it strikes me that the word "Section" is pretty good, for the reason that it implies being a part

of, that is, a "Section" of the American Society of Civil Engineers, and gives it the same standing as, you might say, the Society itself. It is a part of the Society. If you say it is a "Branch" or an "Association", you would think of it as independent, that is, any action taken by a "Branch" or by an "Association" would be considered as independent. Other engineers in other parts of the country might think: "Well, that is an independent organization." They would not consider it as authentic, as being a "Section." That is, if it were understood as being a "Section" of the American Society of Civil Engineers, it would have more weight. I like the word "Section."

Mr. Moritz.—Mr. Chairman, that is just the point. None of these sections has any authority to take any action that will bind the Parent Society, but, if it is a Section, it will take—practically take—final action on some particular work. Of course, that also has to be approved by the Society as a whole, but that just brings out the objection to "Section" more than ever.

Mr. Houston.—The local organization does have authority to act for the parent organization on all local matters, and in that sense we are a part of the parent organization. We are cautioned not to take any action with regard to National matters, but we are at liberty to take action and bind the Society, as I understand it, in regard to local matters.

Chairman Jaycox.—Any further remarks?

Secretary Hinman.—I am in favor of maintaining the present name. I believe that all these local organizations are purely and simply Associations of members. That was the idea when the Colorado Association was organized, that the members in Colorado would associate for the furtherance of social and professional benefits; and, inasmuch as there are at present twenty-one Associations, that fact is quite sufficient for leaving their names just as they are. The local organizations of the Electrical Engineers and the Mining Engineers are called "Sections", and, among outsiders, if the name is a little different, I believe it creates a distinction from simply the term "engineers." The outsider does not generally stop to consider whether engineers are civil, electrical, or mechanical, and, aside from that distinguishing name, I believe the word "Association" as distinguished from "Section" would be advisable and beneficial. I recommend the retention of the name.

Mr. Field.—In order to get an expression of this meeting, I suggest that we be called on to designate which of the three names we prefer.

Chairman Jaycox.—Does the seconder consent to the withdrawal of the motion?

Mr. Houston.—Yes.

CHAIRMAN JAYCOX.—Gentlemen, in order to get an expression of opinion, I will ask for a show of hands, so that the sequence of the action will be taken by numbers. All in favor of the retention of the word "Association", please raise their hands.

SECRETARY HINMAN.—Six.

CHAIRMAN JAYCOX.—Those in favor of the designation "Section" will please raise their hands.

SECRETARY HINMAN.—Five.

CHAIRMAN JAYCOX.—Those in favor of the word "Branch" will please raise their hands.

SECRETARY HINMAN.—One.

CHAIRMAN JAYCOX.—The order of sequence is "Association", "Section", "Branch." What is the next item, Mr. Secretary?

SECRETARY HINMAN.—The next item is "Policy. Relations of Local Sections to the American Society of Civil Engineers."

"*Reports.*—Local Sections should transmit promptly to the Secretary of the American Society of Civil Engineers an abstract of such proceedings of their meetings as may be deemed of general interest," etc.

In the paragraph headed "New Members," the Committee refers to the fact that the local Section should seek in all proper ways to increase the membership; that it "should investigate carefully the qualifications of any candidate for admission to the Society about whom the Board of Direction desires information, and make an official report at the request of the Secretary of the Society."

CHAIRMAN JAYCOX.—This is a new procedure on the part of the Parent Society. At present the recommendation of the endorsers of an applicant is the only information gained by the Board of Direction with reference to his qualifications. This means that the Local Association, when an applicant is within its jurisdiction, shall make a report to the Board of Direction for examination of his qualifications before the Board of Direction takes final action on the application.

SECRETARY HINMAN.—Is it your idea that they shall take final action? This reads that it shall.

CHAIRMAN JAYCOX.—That is changed. Are you in favor of such change being made? That is the only question for us in reference to this matter. In order to expedite matters and proceed as rapidly as we can, all who are in favor of having the application of an applicant referred to the Local Association, please manifest it by the sign of hands.

The question is unanimously adopted. Now, the next one.

SECRETARY HINMAN.—The next paragraph is:

"*Restriction in Activity.*—Local Sections should refrain from all action on matters of a National character, or which might possibly affect the general interests of the Profession, without the approval of the Board of Direction, and should report to the Secretary any local matter which might affect or interest the Society or Profession at large."

CHAIRMAN JAYCOX.—Any remarks to be made on that section? If not, all in favor of its approval will please manifest it by the usual sign. Are there any votes to the contrary? If not, it will receive the approval of the Association.

SECRETARY HINMAN.—The next paragraph is:

"*Ethics.*—Local Sections shall insist on the observance of the Code of Ethics adopted by the American Society of Civil Engineers, and support the Society in matters relating to it. Charges made by any member of the Society that a member of the Section has violated this code should be promptly investigated by a committee of the Section, and its findings officially reported to the Board of Direction."

MR. HOUSTON.—I move its approval.

CHAIRMAN JAYCOX.—I just wish to call your attention to the fact that this paragraph requires that the finding shall be promptly investigated by a committee, but it does not say that the Local Association should approve or disapprove the finding of the committee. I think that, before being sent to the Society, it should be approved by the Local Association. Shall the suggestion of the Chair be inserted in the paragraph on "Ethics"?

MR. FIELD.—Yes, sir, it should.

CHAIRMAN JAYCOX.—As amended, are you ready to vote on the paragraph?

MR. HOUSTON.—Do I understand that in this case the matter should be brought publicly before the whole Association, and the findings approved or disapproved? It seems to me that that would not be very good policy until it had been brought before the Board of Direction, and there might be cases where that would be somewhat embarrassing, if the Board of Direction did not support the finding.

SECRETARY HINMAN.—I don't believe that is the intention. It says that the Section shall appoint a committee to investigate, and that that committee shall report to the Parent Society. I don't believe it would be brought before the meeting.

CHAIRMAN JAYCOX.—My idea is that it should be. We don't know what the action of the committee should be.

MR. HOUSTON.—Until this gets to the Parent Society it should not be made public.

CHAIRMAN JAYCOX.—Only before the Association.

MR. HOUSTON.—I don't believe it should be brought before the Association until it has been brought before the Parent Society.

There might be a great deal of injustice in that, depending on who comprised the committee. In appointing this committee, the Association should use its best judgment in the matter, and should be willing to trust to its findings and present its report to the Board of Direction before it comes publicly before the Association.

Mr. Moritz.—I would like to ask what will the Board of Direction do with this report? There is nothing said about that.

Chairman Jaycox.—Gentlemen, I have examined the report for the purpose of talking of these matters, and think that, although we might appoint an excellent committee, and its members might have the best of intention as to their duties, they might not get accurate information or all that might be known, and the Association should have the privilege of reviewing the finding of its own committee before it becomes a part of our proceedings and is transmitted to the Board of Direction.

Secretary Hinman.—I think that the committee appointed by the Association acts for the Parent Society more than for the Association. In other words, the Parent Society uses the members of the Association for its convenience in this investigation, rather than send anybody else to investigate the matter, and that that committee should report to the Board of Direction, and then the Board can further instruct the Society as to what should be done. I believe, with Mr. Houston, that a man's reputation or profession might be very seriously and erroneously damaged by making anything public. Anything you make public, unless you are absolutely sure of it—and I don't see how we could be without investigation—might damage a man's career beyond reparation.

Mr. Field.—As to the matter of publicity, your man is investigated; the committee is appointed; that is publicity enough. Now give him some sort of a show. Let everybody in the Association—in his district—hear this report and say what they think about it. I think a man will get a much better hearing, and the Local Association will know much better what it is doing.

Chairman Jaycox.—Any further remarks on the amendment made by the Chair?

Mr. Houston.—How are the charges to be made in a case of this kind—before the Local Association or before the Board of Direction, as provided in the present Constitution and By-Laws of the Society?

Chairman Jaycox.—The matter comes before the Board of Direction.

Mr. Houston.—As it ordinarily did—as I think it did—by a petition of ten?

Chairman Jaycox.—Yes.

Mr. Houston.—Then the petition does not come before the Society, but the Board of Direction notifies the Local Association to

appoint a committee for investigation. They need not mention who is to be investigated; simply appoint the committee, and then the name could be furnished by the Board of Direction. I don't think there should be any publicity in this matter until the local committee has heard the evidence and placed it before the Board of Direction. That more nearly corresponds with our present arrangement, which seems to have been very satisfactory.

Mr. Black.—Mr. Chairman, all such investigations as this call for evidence. That is what you have to draw your conclusion from, and it would be difficult to present all the evidence before the whole Association. I would be much more willing to trust to the judgment of a committee composed of two or three reputable men in our community, and let them render the decision than to hear only the evidence presented to our meeting as a whole.

Chairman Jaycox.—Gentlemen, I do not want to insist on this, but we do not know what kind of a committee will be appointed. Any member of the Society can bring any member of an Association to the carpet. The matter of itself is a rather harsh way of doing business, it assails a man's reputation, and is referred to the Association to find out whether those charges—perhaps made by a member in New York—are true. The appointment of the committee is done by the Association. I can very easily imagine a condition of affairs where a committee would not be unbiased, even in our own Association. I don't say that it would be, but it could be. If an action of that kind is taken, we don't know anything about the evidence; it goes to three men. What their report may be on one of our fellow-members is not reviewable by our own Association. I think it is rather drastic. Any further remarks?

Mr. A. N. Miller.—The number of men on the committee should be increased to six instead of three.

Chairman Jaycox.—The number on the committee, I think, will be entirely within the hands of the Local Association.

Mr. J. W. Deen.—I like this idea of referring these applications for membership to the Local Associations.

Chairman Jaycox.—That has been passed on, Mr. Deen.

Mr. Deen.—And we have not had enough publicity, so far as concerns the people getting into the Society. I know men in this Society—not in our own local organization but in the United States—who have gone a long way from home to get their endorsers, in order to get into the Society, and the people at home did not know it until they were members—did not examine the publications, and it slipped by them.

Chairman Jaycox.—Mr. Deen, we have passed that order, and have approved the reference to the local organization for the members in our district.

Mr. Deen.—Well, I like that idea.

Chairman Jaycox.—Gentlemen, the question will be first on whether or not the Local Association wants to be informed of the finding of the committee to which a matter of this kind is referred. All in favor of incorporating that idea in this section will raise their hands; those opposed. It is an even vote so far, gentlemen. That being the case, the amendment is not incorporated. Now the question is on the paragraph headed "Ethics"; shall it be approved by the Association? Manifest your approval by vote, please.

The section is approved.

Secretary Hinman.—The next paragraph is:

"*Grievances.*—Local Sections should endeavor to protect the professional reputation of their members by investigating and publishing the facts of any case in which their integrity or competency appears to have been unfairly or maliciously misrepresented, and should report the circumstances and the action taken to the Board of Direction."

Chairman Jaycox.—Any discussion or remarks on this section? All in favor of the approval of this section, please say "aye". It is carried.

Secretary Hinman.—The next heading is: "Relations of Local Sections to Sections of other National Societies, to Local Engineering Clubs, and to All Engineers". The first paragraph is:

"*Co-operation.*—Local Sections should adopt a broad view of their relations to the Engineering Profession, and recognize the advantage and necessity of co-operation with other engineering organizations in all ways that may strengthen the position of engineers, develop social relations among them, or tend to establish correct principles and practice."

That paragraph simply encourages co-operation.

Chairman Jaycox.—There will be no objection; all are in favor of it.

Secretary Hinman.—(Reading).

"*Local Societies.*—Local Sections should affiliate with existing Engineering Societies, and, where none exist and the conditions are favorable, they should encourage their formation. All engineers should be considered as eligible for membership in such Societies, but only those whose attainments correspond at least to the requirements for Associate Membership in the American Society of Civil Engineers should have a vote. It is desirable that not more than 20% of the membership of these Societies should be admitted as Associates, who, although not engineers, have interests in common with them. Members of a Section of the American Society of Civil Engineers should vote in such Societies only as individuals, and not as a body."

Chairman Jaycox.—The purport of this paragraph is that there should be formed a local society, not of the American Society of Civil

Engineers, but of all and any engineers who might be taken under the Constitution of that Society, and that the American Society members should go in as any other members of that Society. That is what it is—and to try to limit it to a qualification for membership which will only include 20% of the membership of that Society equal in grade and qualification to our Associate Members—I don't see how it is possible to do it.

MR. DEEN.—It is quite a complicated proposition.

MR. HOUSTON.—Mr. President, one of the provisions of the branches of the Canadian Society of Civil Engineers is that engineers who are not members of the Canadian Society may be admitted as associates of the branch. They are not members of the Parent Society at all, but are admitted by vote and on the payment of dues, but have no vote in the Society, and that, to a certain extent, covers the same ground that this would. It provides for a society for those men who are not members of the parent organization, and still keeps the identity of the Local Association or Section of the American Society.

CHAIRMAN JAYCOX.—But, Mr. Houston, your basis is a section of the Canadian Society. This is an organization of a society without regard to any one else, just the same as if all the engineers of the City of Denver should get together and form a society. We have a society which could serve as a base now—the Colorado Society of Civil Engineers.

MR. HOUSTON.—The Parent Society wants such societies to have a certain standing, namely, equal to that of our Associate Members.

CHAIRMAN JAYCOX.—Only 20% of the membership. When that is filled up, nobody else can join.

MR. A. E. PALEN.—Does that 20% apply to the associate members—the associates of the Local Society?

CHAIRMAN JAYCOX.—Twenty per cent. of the members of the Society. Now, remember, that is the Local Society. It is not a Section or Association of the American Society. It is a new society entirely. And this section limits that. Not more than 20% shall be admitted who do not have qualifications of an Associate Member of our Society.

MR. PALEN.—My interpretation of the reading of it was that not more than 20% of this Local Society should be men who are associated in engineering matters, but are not engineers. They are below the qualifications of Associate Members of the Parent Society.

CHAIRMAN JAYCOX.—But that limits it. As soon as you get 20%, you cannot have a larger society of those men. You understand the question, gentlemen?

MR. HOUSTON.—I don't think I understand it yet. Does it infer that we are to organize as a section of the American Society and affiliate with these——

CHAIRMAN JAYCOX.—No, we are simply to go in as any other member would go into this Society, and encourage the organization of an independent organization or local organization of the American Society.

MR. HOUSTON.—We could have both, then?

CHAIRMAN JAYCOX.—Yes.

SECRETARY HINMAN.—This paragraph refers to places where organizations are not already in existence—that they should be encouraged by members of the American Society who may live in that locality. If there were no engineering organizations in Denver, for instance, the local members of the American Society of Civil Engineers should encourage the organization of engineers. Assuming that such a society were formed and had 100 members, then this paragraph recommends that 20 of those men need not necessarily be engineers, but may be men who have interests in common, and those 20 members may be admitted, but shall not have a vote; that only those above those 20 men—the other 80, in other words—shall have a vote, and their requirements shall be equal to that of Associate Members in the American Society, but the remaining 20 need not necessarily have that requirement. It seems to me that this paragraph is perhaps getting somewhat dictatorial. I don't see why the American Society should do any more than to further the organization. It should be left to such a local organization to establish its own rules and regulations, and I believe that the American Society members in the locality could be depended on to form proper rules.

MR. FIELD.—When such a society is formed, the members of the American Society of Civil Engineers should endeavor to maintain for that society a certain standard, and not have a society in their vicinity calling itself a society of engineers when its members really have no standing as such; and it is our duty, when such a society is formed, to try to see that it has a sufficient number of engineers to justify its members in calling themselves engineers, and for that reason only 20% of such members should be below the grade of Associate Member.

CHAIRMAN JAYCOX.—I think it is a good object to work for, but I don't think that you can organize a local society with that as a limit.

MR. FIELD.—No, we cannot control that, but we can exercise our good offices with that idea in view.

MR. BLACK.—I take it that the spirit of this paragraph is that we should co-operate with the other branches of engineering, and do our bit to raise the standard of all engineers, whether electrical, mechanical, civil, or mining, whatever they may be, as we have a standard in our Society, and that would encourage us to associate with other branches of engineering, and do what we could to raise the standard, which, I think, is what all would want to do under those circumstances.

SECRETARY HINMAN.—Mr. President, this paragraph is not applicable to our locality. We have organizations of electricals and mining; we have the Colorado Scientific Society, and the previous paragraph on "Co-operation" is applicable. I move that the discussion of this paragraph on "Local Societies" be discontinued.

CHAIRMAN JAYCOX.—And not make any expression of opinion about it?

SECRETARY HINMAN.—No.

CHAIRMAN JAYCOX.—That is evading the question.

MR. MORITZ.—I move that the section be eliminated.

MR. HOUSTON.—I move that it be adopted.

(The motion for adoption was duly seconded.)

CHAIRMAN JAYCOX.—It is moved that the section meets with our approval. All in favor of this motion show hands. Those opposed to this motion. The section is approved.

SECRETARY HINMAN.—The next paragraph is:

"*Engineers.*—The Local Sections should endeavor to promote friendship among engineers, regardless of their connection with any Society, and to extend to them professional support. They should oppose any tendency among engineers to pay inadequate compensation to engineers whom they employ, on account of the experience to be gained by the employment, and should discourage in all proper ways persons who assume the functions of engineers without possessing their recognized qualifications."

MR. FIELD.—I move its adoption.

(The motion was duly seconded and carried unanimously.)

SECRETARY HINMAN.—The next section is "Relations of Local Sections to the Public." This means that we should educate the public to the fact that engineers exist.

"*Public Affairs.*—Local Sections should volunteer judicious and carefully considered advice on public matters involving engineering questions, but this advice must be above any suspicion of connection with personal advantage. It is undesirable that there should be any encroachment on the practice of individual engineers or participation in personal or partisan politics, and it is of the highest importance to the Profession that engineers should realize and accept their duties as citizens of the community in which they live."

MR. HOUSTON.—I move its adoption.

(The motion was seconded by Mr. Field and carried.)

CHAIRMAN JAYCOX.—That is one of the hardest things that can be done. It is very well to have an opportunity of doing, but in order to get engineers interested in public affairs and to get them—their opinions, actions, and reports—before the public, is a different job. The next one, Mr. Secretary.

SECRETARY HINMAN.—(Reading.)

"*State Affairs.*—Local Sections need not be confined to State lines, but they should endeavor to arrange some organization of the engineers of the States within their territory, for the purpose of exerting influence in the legislation of the State and the administration of its affairs wherever engineering principles or practices are involved."

CHAIRMAN JAYCOX.—We have already done that, so there is no necessity for a ruling. Any other questions in reference to the affairs of this Society or Association are now in order.

MR. HOUSTON.—The parent organization should rebate to each section some portion of the dues which its members pay. This would not hamper the parent organization in its financial condition at present. The income from dues was about $89 000 last year.

CHAIRMAN JAYCOX.—More than that.

MR. HOUSTON.—That is from ordinary dues. I think I am right about that—about $89 000—and the income from other sources, outside of the sale of some bonds, was more than $100 000. If the parent organization should rebate $2 or $3 per member to the Local Association, it would be a great financial help to the latter, and as the Society has only 8 000 members, and it is probable that not more than 50% of them would be in local associations—suppose there were 50%—that would only make $8 000 out of the $89 000. That is, it would reduce the income of the parent organization from ordinary dues to $80 000 instead of $89 000 as last year, with the balance which is obtained, of $100 000 or more, from other sources. The Parent Society would not feel this—could easily spare this amount of money, and it would encourage the organization of local sections, and all who have had anything to do with local associations know that the financial side is hard to handle. I move that it be the sense of this local organization that the American Society of Civil Engineers rebate some portion of the dues which are paid by local members of the Section or Association, for the benefit of the local Section.

CHAIRMAN JAYCOX.—Is there a seconder to Mr. Houston's motion?

(The motion was duly seconded.)

CHAIRMAN JAYCOX.—Now, a discussion on this motion.

SECRETARY HINMAN.—I believe that the Parent Society should make this rebate. We pay $15 dues, and we get the *Proceedings* and a great many other benefits, perhaps, from the American Society of Civil Engineers. All the other National engineering societies support their local associations to some extent. For instance, all the local sections of the Electrical Engineers have a flat amount of $100, and, in addition, $1.25 for each member within a radius of 60 miles. The Mining Engineers have a rebate of one-quarter of the dues paid by all the members in the section, with a proviso that such amount shall not exceed $250. This Association has a membership that fluctuates between 80 and 90, perhaps nearer 85 than 90. That means an income

of from $170 to $180 a year, and if we squeeze pretty hard—are very careful and nail people to the cross—we get out with an expenditure of from $150 to $160 a year. We are deprived in a great many cases of things we would like to have by reason of small financial income. I believe that the American Society of Civil Engineers should rebate to each of its Local Associations not less than $3 per member. If it is necessary to define certain limits of territory, so that two Associations may not get rebate for the same member, well and good. The Colorado Association would be willing to agree to remain within the limits of Colorado and such other portions of District No. 10 as would be large enough for the establishment of an association. I would suggest that Mr. Houston's motion be a little more specific in stating that not less than $3 per member be returned. That is less than 25%, and I would place that as an amendment.

Now, our Constitution provides that our dues shall not exceed $5. The San Francisco Association has dues of $5, and they now have several thousand dollars to their credit. They have made investments in different bonds. They carry a balance in bank of more than $300 nearly all the time. They provide their meetings with quite a bit of entertainment during their dinners, cabaret features and one thing and another—which evidently, from their reports, are very interesting and bring out a large membership. I don't believe that our members see the reports of the San Francisco Association, but the attendance at their meetings is as large as at most meetings of the American Society in New York, and I believe it is largely a matter of financial ability to conduct the meetings.

(The amendment was duly seconded.)

Chairman Jaycox.—We will incorporate that without objection. Gentlemen, any further remarks on this motion? The motion is that the Parent Society be requested to take into consideration a rebate of not less than $3 per member for the benefit of the Local Associations as organized. Any further remarks? If not, all in favor of the adoption of the motion, please say "aye"; contrary, "no"; the motion is carried. Any further business?

Gentlemen, that concludes the business to which our attention has been called by the Secretary, and of which we are to make report to the Parent Society. If there are no remarks on this subject, we will proceed with our ordinary local business.

ANNOUNCEMENTS

The House of the Society is open from 9 A. M. to 10 P. M., every day, except Sundays, Fourth of July, Thanksgiving Day, and Christmas Day.

FUTURE MEETINGS

November 7th, 1917.—8.30 P. M.—A regular business meeting will be held, and two papers will be presented for discussion, as follows: "The Subsidence of Muck and Peat Soils in Southern Louisiana and Florida", by Charles W. Okey, Assoc. M. Am. Soc. C. E.; and, "Pulsations in Pipe Lines, as Shown by Some Recent Tests", by H. C. Vensano, M. Am. Soc. C. E.

Mr. Okey's paper was printed in *Proceedings* for September, 1917, and Mr. Vensano's paper is printed in this number of *Proceedings*.

November 21st, 1917.—8.30 P. M.—At this meeting two papers will be presented for discussion, as follows: "The Hell Gate Arch Bridge and Approaches of the New York Connecting Railroad Over the East River in New York City", by O. H. Ammann, M. Am. Soc. C. E.; and, "Stress Measurements on the Hell Gate Arch Bridge," by D. B. Steinman, Assoc. M. Am. Soc. C. E.

These papers are printed in this number of *Proceedings*.

LIST OF NOMINEES FOR THE OFFICES TO BE FILLED AT THE ANNUAL ELECTION, JANUARY 16th, 1918

The following list of nominees for the offices to be filled at the Annual Meeting, January 16th, 1918, received from the Nominating Committee, was presented to the Board of Direction at its meeting on October 9th, 1917. The list has already been mailed to all Corporate Members:

For President, to serve one year:

Arthur N. Talbot, Urbana, Ill.

For Vice-Presidents, to serve two years:

John F. Coleman, New Orleans, La.
Nelson P. Lewis, New York City

For Treasurer, to serve one year:

George W. Tillson, Brooklyn, N. Y.

For Directors, to serve three years:

George W. Goethals, New York City........District No. 1.
Andrew M. Hunt, New York City...........District No. 1.
Samuel Tobias Wagner, Philadelphia, Pa....District No. 4.
Edward E. Wall, St. Louis, Mo.............District No. 9.
Milo S. Ketchum, Boulder, Colo............District No. 10.
Harry Hawgood, Los Angeles, Cal...........District No. 11.

SEARCHES IN THE LIBRARY

In January, 1902, the Secretary was authorized to make searches in the Library, upon request, and to charge therefor the actual cost to the Society for the extra work required. Since that time many searches have been made, and bibliographies and other information on special subjects furnished.

The resulting satisfaction, to the members who have made use of the sources of the Society in this manner, has been expressed frequently, and leaves little doubt that if it were generally known to the membership that such work would be undertaken, many would avail themselves of it.

The cost is trifling compared with the value of the time of an engineer who looks up such matters himself, and the work can be performed quite as well, and much more quickly, by persons familiar with the Library.

In asking that such work be undertaken, members should specify clearly the subject to be covered, and whether references to general books only are desired, or whether a complete bibliography, involving search through periodical literature, is desired.

It sometimes happens that references are found which are not readily accessible to the person for whom the search is made. In that case the material may be reproduced by photography, and this can be done for members at the cost of the work to the Society, which is small. This method is particularly useful when there are drawings or figures in the text which would be very expensive to reproduce by hand.

A list of 989 bibliographies made in the Library, giving the cost of each, was published in Vol. LXXX of *Transactions*.

Since October 1st, 1916, the Library of the American Society of Civil Engineers has ceased to exist, as such, having been merged with the Libraries of the Mining, Mechanical, and Electrical Engineers, and become a part of the Library of the United Engineering Society. There were 67 000 accessions, which were not duplicates, turned over to that Library.

Hereafter, therefore, requests for searches should be addressed to the Librarian, United Engineering Society, 29 West 39th Street, New York City.

PAPERS AND DISCUSSIONS

Members and others who take part in the oral discussions of the papers presented are urged to revise their remarks promptly. Written communications from those who cannot attend the meetings should be sent in at the earliest possible date after the issue of a paper in *Proceedings*.

All papers accepted by the Publication Committee are classified by the Committee with respect to their availability for discussion at meetings.

Papers which, from their general nature, appear to be of a character suitable for oral discussion, will be published as heretofore in *Proceedings*, and set down for presentation to a future meeting of the Society, and on these oral discussions, as well as written communications, will be solicited.

All papers which do not come under this heading, that is to say, those which, from their mathematical or technical nature, in the opinion of the Committee, are not adapted to oral discussion, will not be scheduled for presentation to any meeting. Such papers will be published in *Proceedings* in the same manner as those which are to be presented at meetings, but written discussions only will be requested for subsequent publication in *Proceedings* and with the paper in the volumes of *Transactions*.

The Board of Direction has adopted rules for the preparation and presentation of papers, which will be found on page 429 of the August, 1913, *Proceedings*.

LOCAL ASSOCIATIONS OF MEMBERS OF THE AMERICAN SOCIETY OF CIVIL ENGINEERS

San Francisco Association, Organized 1905.

J. D. Galloway, President; E. T. Thurston, Secretary-Treasurer, 57 Post Street, San Francisco, Cal.

The San Francisco Association of Members of the American Society of Civil Engineers holds regular bi-monthly meetings, with banquet, and weekly informal luncheons. The former are held at 6 P. M., at the Palace Hotel, on the third Tuesday of February, April, June, August, October, and December, the last being the Annual Meeting of the Association.

Informal luncheons are held at 12.30 P. M., every Wednesday, and the place of meeting may be ascertained by communicating with the Secretary.

The by-laws of the Association provide for the extension of hospitality to any member of the Society who may be temporarily in San Francisco, and any such member will be gladly welcomed as a guest.

Colorado Association, Organized 1908.

Robert Follansbee, President; L. R. Hinman, Secretary-Treasurer, 1400 West Colfax Avenue, Denver, Colo.

The meetings of the Colorado Association of Members of the American Society of Civil Engineers (Denver, Colo.) are held on the second Saturday of each month, except July and August. The hour and place of meeting are not fixed, but this information will be furnished on application to the Secretary. The meetings are usually preceded by an informal dinner. Members of the American Society of Civil Engineers will be welcomed at these meetings.

Weekly luncheons are held on Wednesdays at 12.30 P. M., at Daniel's and Fisher's.

Visiting members are urged to attend the meetings and luncheons.

(Abstract of Minutes of Meeting)

June 9th, 1917.—The Ninth Annual Meeting was called to order at the Denver Athletic Club; President Jaycox in the chair; L. R. Hinman, Secretary; and present, also, 13 members.

The minutes of the meeting of May 12th, 1917, were read and approved.

President Jaycox read the Annual Address.

The Secretary-Treasurer presented the Annual Reports, and, on motion, duly seconded, these were ordered filed.

Chairman G. N. Houston, for the Auditing Committee, reported that the financial affairs of the Association were correctly stated in the Treasurer's report.

The report of the Nominating Committee was received, and Messrs. G. N. Houston, and E. A. Moritz were appointed Tellers to canvass the ballots for Officers.

On the report of the Tellers the following officers were declared elected: President, Robert Follansbee; Vice-President, W. C. Huntington; and Secretary-Treasurer, L. R. Hinman.

The proposed amendment to the By-laws relative to the appointment of Technical and Non-Technical Committees was read, and, on motion, duly seconded, was adopted as "Article VIII—Committees."

The Secretary read a letter from Chas. Warren Hunt, Secretary of the Society, requesting that members of the Association discuss the report of the Committee on Relations of Local Associations which appeared in *Proceedings* for May, 1917. Each section of the report was read and discussed, a vote being taken for or against its adoption or revision.*

Adjourned.

Atlanta Association, Organized 1912.

Paul H. Norcross, President; Thomas P. Branch, Secretary-Treasurer, Georgia School of Technology, Atlanta, Ga.

The Association holds its meetings at the University Club, Atlanta, Ga. Regular monthly luncheon meetings are held to which visiting members of the Society are always welcome.

Baltimore Association, Organized 1914.

Mason D. Pratt, President; Charles J. Tilden, Secretary-Treasurer, The Johns Hopkins University, Baltimore, Md.

Cleveland Association, Organized 1914.

W. J. Watson, President; George H. Tinker, Secretary-Treasurer, 516 Columbia Building, Cleveland, Ohio.

Detroit Association, Organized 1916.

T. A. Leisen, President; Clarence W. Hubbell, Secretary, 2348 Penobscot Building, Detroit, Mich.

The regular meetings of the Association are held on the second Friday of December, April, and October, the last being the Annual Meeting.

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* See page 590.

District of Columbia Association, Organized 1916.

A. P. Davis, President; John C. Hoyt, Secretary-Treasurer, U. S. Geological Survey, Washington, D. C.

Duluth Association, Organized 1917.

F. E. House, President; Walter G. Zimmermann, Secretary, Wolvin Building, Duluth, Minn.

The regular meetings of the Association are held monthly. The time and place of meeting are not fixed, but this information will be furnished on application to the Secretary. The Annual Meeting is held on the third Monday of May.

Illinois Association, Organized 1916.

C. F. Loweth, President, Chicago, Ill.

The regular meetings of the Association are held on the second Monday of March, June, September, and December, the last being the Annual Meeting. The hour and place of meeting are not fixed, but this information will be furnished on application to the President.

Louisiana Association, Organized 1914.

W. B. Gregory, President; Charles W. Okey, Secretary, Tulane University, New Orleans, La.

The regular meetings of the Association are held at The Cabildo, New Orleans, La., on the first Monday of January, April, July, and October.

Nebraska Association, Organized 1917.

Frank T. Darrow, President; Homer V. Knouse, Secretary-Treasurer, 115 City Hall, Omaha, Nebr.

Regular meetings of the Association are held on the first Saturday of each month, except July and August, and at such places as may be appointed from time to time by the Executive Committee. The Annual Meeting is held in Lincoln, Nebr., on the second Friday in January.

It is probable that frequent luncheons will be held in Omaha, in addition to the monthly meetings, at which visiting members will be welcomed. The place of meeting may be ascertained by communicating with the Secretary.

Northwestern Association, Organized 1914.

George L. Wilson, President; Ralph D. Thomas, Secretary, 508 South First Street, Minneapolis, Minn.

Philadelphia Association, Organized 1913.

Samuel T. Wagner, President; C. W. Thorn, Secretary, 1313 South Broad Street, Philadelphia, Pa.

The regular meetings of the Association are held at the Engineers' Club of Philadelphia, 1317 Spruce Street, on the First Monday in January, April, and October, the last being the Annual Meeting.

Portland, Ore., Association, Organized 1913.

J. P. Newell, President; J. A. Currey, Secretary, 194 North 13th Street, Portland, Ore.

St. Louis Association, Organized 1914.

J. A. Ockerson, President; Gurdon G. Black, Secretary-Treasurer, 34 East Grand Avenue, St. Louis, Mo.

The meetings of the Association are held at the Engineers' Club Auditorium. The Annual Meeting is held on the fourth Monday in November. The time of other meetings is not fixed, but this information will be furnished on application to the Secretary.

San Diego Association, Organized 1915.

W. J. Gough, President; J. R. Comly, Secretary-Treasurer, 4105 Falcon Street, San Diego, Cal.

Seattle Association, Organized 1913.

Joseph Jacobs, President; Carl H. Reeves, Secretary-Treasurer, 444 Henry Building, Seattle, Wash.

The regular meetings of the Association are held at 12.15 P. M., on the last Monday of each month, at The Frye Hotel.

(Abstract of Minutes of Meeting)

August 27th, 1917.—The meeting was called to order at 12.15 P. M., at the Hotel Frye; President Jacobs in the chair; O. P. M. Goss acting as Secretary; and present, also, 11 members.

The minutes of the meeting of July 30th, 1917, were read and approved.

Mr. John L. Hall brought up the question of the payment of the express charge on volumes of *Transactions* presented by the Society to the Library of the Pacific Northwest Society of Engineers. On motion, duly seconded, it was voted that hereafter the Association should pay this charge.

Relative to a committee recently appointed by the Chamber of Commerce to investigate the Seattle Port Commission activities and the Port of Seattle, President Jacobs expressed regret that, considering the engineering matters involved, no engineer had been appointed on this Committee, and suggested that the Associated Engineering Societies call the attention of the President of the Chamber of Commerce to this matter. After discussion by Messrs. Hussey, Grant, and Hall, it was decided, on motion, duly seconded, to refer the matter to the Council of the Associated Engineering Societies with the suggestion that it take any action thereon which it may deem advisable.

Mr. A. H. Fuller presented the report of the Committee on Relations with the Parent Society to which had been submitted the report to the Board of Direction of the Committee on relations of Local Associations to the Society, and, after brief discussion, it was decided, on motion, duly seconded, that the Committee be authorized to edit the report, and that President Jacobs should then forward it to the Society with a letter setting forth arguments in favor of its adoption.*

Adjourned.

Southern California Association, Organized 1914.

H. Hawgood, President; Wilkie Woodard, Secretary, 114 West 51st St., Los Angeles, Cal.

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* See page 587.

The Southern California Association of Members of the American Society of Civil Engineers (Los Angeles, Cal.) holds regular bi-monthly meetings, with banquet, at Hotel Clark, on the second Wednesday of February, April, June, August, October, and December, the last being the Annual Meeting of the Association.

Informal luncheons are held at 12.15 P. M. every Wednesday, and the place of meeting may be ascertained from the Secretary.

The by-laws of the Association provide for the extension of hospitality to any member of the Society who may be temporarily in Los Angeles, and any such member will be gladly welcomed as a guest at any of the meetings or luncheons.

Spokane Association, Organized 1914.

J. C. Ralston, President; B. J. Garnett, Secretary, City Hall, Spokane, Wash.

The regular meetings of the Association are held on the second Friday of each month, except July and August. The hour and place of meeting are not fixed, but this information will be furnished on application to the Secretary.

Visiting members are invited to attend the meetings and luncheons.

Texas Association, Organized 1913.

John B. Hawley, President; J. F. Witt, Secretary, Dallas, Tex.

Utah Association, Organized 1916.

George L. Swendsen, President; H. S. Kleinschmidt, Secretary-Treasurer, 306 Dooly Building, Salt Lake City, Utah.

The Annual Meeting of the Association is held on the first Wednesday in April. The time of other meetings is not fixed, but this information will be furnished on application to the Secretary.

MINUTES OF MEETINGS OF SPECIAL COMMITTEES TO REPORT UPON ENGINEERING SUBJECTS

Special Committee on Materials for Road Construction

August 27th, 1917.—The meeting was called to order at 9.40 A. M., at the House of the Society. Present, George W. Tillson (Chairman *pro tem.*), Charles J. Tilden, and Arthur H. Blanchard (Secretary).

The minutes of the meeting of July 9th, 1917, were read and approved.

On motion, duly seconded, it was agreed that the section of the 1918 Report covering Definitions shall consist of the list of definitions contained in the 1915 Report as amended at this meeting.

On motion, duly seconded, it was ordered that each Sub-Committee shall forward, on or before October 1st, 1917, to each member of the Committee, a copy of the final draft of its report, the Secretary to receive two copies of each report.

On motion, duly seconded, it was decided that the next meeting of the Committee shall be held at 9.30 A. M., on October 15th, 1917, at the House of the Society.

On motion, duly seconded, it was agreed that a meeting of the Committee be held at 9.30 A. M. on October 22d, 1917, at the House of the Society.

On motion, duly seconded, it was decided that the order of the day at the meetings of October 15th and 22d, 1917, be the final reports of the several Sub-Committees.

PRIVILEGES OF ENGINEERING SOCIETIES EXTENDED TO MEMBERS OF THE AMERICAN SOCIETY OF CIVIL ENGINEERS

Members of the American Society of Civil Engineers will be welcomed by the following Engineering Societies, both to the use of their Reading Rooms, and at all meetings:

American Institute of Electrical Engineers, 25 West Thirty-ninth Street, New York City.

American Institute of Mining Engineers, 25 West Thirty-ninth Street, New York City.

American Society of Mechanical Engineers, 25 West Thirty-ninth Street, New York City.

Assoçiacão dos Engenheiros Civis Portuguezes, Lisbon, Portugal.

Australasian Institute of Mining Engineers, Melbourne, Victoria, Australia.

Boston Society of Civil Engineers, 715 Tremont Temple, Boston, Mass.

Brooklyn Engineers' Club, 117 Remsen Street, Brooklyn, N. Y.

Canadian Society of Civil Engineers, 176 Mansfield Street, Montreal, Que., Canada.

Civil Engineers' Society of St. Paul, St. Paul, Minn.

Cleveland Engineering Society, Chamber of Commerce Building, Cleveland, Ohio.

Cleveland Institute of Engineers, Middlesbrough, England.

Dansk Ingeniorforening, Amaliegade 38, Copenhagen, Denmark.

Detroit Engineering Society, 46 Grand River Avenue, West, Detroit, Mich.

Engineering Association of Nashville, Commercial Club Building, Nashville, Tenn.

Engineers and Architects Club of Louisville, 1412 Starks Building, Louisville, Ky.

Engineers' Club of Baltimore, 6 West Eager Street, Baltimore, Md.

Engineers' Club of Kansas City, E. B. Murray, Secretary, 920 Walnut Street, Kansas City, Mo.

Engineers' Club of Minneapolis, 17 South Sixth Street, Minneapolis, Minn.

Engineers' Club of Philadelphia, 1317 Spruce Street, Philadelphia, Pa.

Engineers' Club of St. Louis, 3817 Olive Street, St. Louis, Mo.

Engineers' Club of Toronto, 96 King Street, West, Toronto, Ont., Canada.

Engineers' Club of Trenton, Trent Theatre Building, 12 North Warren Street, Trenton, N. J.

Engineers' Society of Northeastern Pennsylvania, 415 Washington Avenue, Scranton, Pa.

Engineers' Society of Pennsylvania, 31 South Front Street, Harrisburg, Pa.

Engineers' Society of Western Pennsylvania, 568 Union Arcade Building, Pittsburgh, Pa.

Florida Engineering Society, J. R. Benton, Secretary, Gainesville, Fla.

Institute of Marine Engineers, The Minories, Tower Hill, London, E., England.

Institution of Civil Engineers, Great George Street, Westminster, S. W., London, England.

Institution of Engineers of the River Plate, Calle 25 de Mayo 195, Buenos Aires, Argentine Republic.

Institution of Naval Architects, 5 Adelphi Terrace, London, W. C., England.

Junior Institution of Engineers, 39 Victoria Street, Westminster, S. W., London, England.

Koninklijk Instituut van Ingenieurs, The Hague, The Netherlands.

Louisiana Engineering Society, State Museum Building, Chartres and St. Ann Streets, New Orleans, La.

Memphis Engineers' Club, Memphis, Tenn.

Midland Institute of Mining, Civil and Mechanical Engineers, Sheffield, England.

Montana Society of Engineers, Butte, Mont.

North of England Institute of Mining and Mechanical Engineers, Newcastle-upon-Tyne, England.

Oregon Society of Civil Engineers, Portland, Ore.

Pacific Northwest Society of Engineers, 803 Central Building, Seattle, Wash.

Rochester Engineering Society, Rochester, N. Y.

Sociedad Colombiana de Ingenieros, Bogota, Colombia.

Sociedad de Ingenieros del Peru, Lima, Peru.

Societe des Ingenieurs Civils de France, 19 rue Blanche, Paris, France.

Society of Engineers, 17 Victoria Street, Westminster, S. W., London, England.

Svenska Teknologforeningen, Brunkebergstorg 18, Stockholm, Sweden.

Tekniske Forening, Vestre Boulevard 18–1, Copenhagen, Denmark.

Vermont Society of Engineers, George A. Reed, Secretary, Montpelier, Vt.

Western Society of Engineers, 1735 Monadnock Block, Chicago, Ill.

ACCESSIONS TO THE UNITED ENGINEERING SOCIETY LIBRARY

(From August 10th to October 1st, 1917)

DONATIONS*

The statements made in these notices are taken directly from the books themselves, and this Society is not responsible for them.

A TEXT-BOOK ON ROOFS AND BRIDGES:

Part II, Graphic Statics. By Mansfield Merriman and Henry S. Jacoby. 4th ed., rev. and enl. N. Y., John Wiley & Sons, Inc.; Lond., Chapman & Hall, Ltd., 1917. 10 + 294 pp., 162 illus., 6 pl., 9 x 6 in., cloth. $2.50.

In addition to a revision and rewriting of the third edition, three new chapters, on Influence Lines for Stresses, Deflection Influence Lines, and Reference Literature of Graphics, have been added. The book contains sixty-two more pages than the third edition, and twenty-four additional drawings.

ELECTRICAL MEASUREMENTS.

By Frank A. Laws. N. Y., McGraw-Hill Book Co., Inc.; Lond., Hill Publishing Co., Ltd., 1917. 13 + 719 pp., 442 illus., 8 x 6 in., cloth. $5.00.

This book is a general treatment of the subject, based on the author's experience at the Massachusetts Institute of Technology, which emphasizes matters of importance to the student of electrical engineering. Intended as a general reference work as well as a text for students. References to more detailed discussions of the various methods are appended to each chapter.

THEORY AND CALCULATIONS OF ELECTRICAL APPARATUS.

By Charles Proteus Steinmetz. N. Y., McGraw-Hill Book Co., Inc.; Lond., Hill Publishing Co., Ltd., 1917. 22 + 480 pp., 227 illus., 9 x 6 in., cloth. $4.00.

The author discusses the most important characteristics of the numerous types of apparatus which have been devised and have found their place in the theory of electrical engineering. The work is a continuation, or rather a part of, his "Theory and Calculation of Alternating Current Phenomena." The matter appearing in earlier editions of that work has now been expanded and distributed into three volumes: "Alternating Current Phenomena", "Electric Circuits", and "Electrical Apparatus." Contents: Speed Control of Induction Motors; Multiple Squirrel Cage Induction Motor; Concatenation; Induction Motor with Secondary Excitation; Single-Phase Induction Motor; Induction Motor Regulation and Stability; Higher Harmonics in Induction Motors; Synchronizing Induction Motors; Synchronous Induction Motor; Hysteresis Motor; Rotary Terminal Single-Phase Induction Motors; Frequency Converter or General Alternating Current Transformer; Synchronous Induction Generator; Phase Conversion and Single-Phase Generation; Synchronous Rectifiers; Reaction Machines; Inductor Machines; Surging or Synchronous Motors; Alternating Current Motors in General; Single-Phase Commutator Motors; Regulating Pole Converter; Unipolar Machines; Review; Conclusion.

ELECTRICAL MACHINERY:

Principles, Operation, and Management. By Terrell Croft. N. Y., McGraw-Hill Book Co., Inc.; Lond., Hill Publishing Co., Ltd., 1917. 318 pp., 302 illus., 8 x 6 in., cloth. $2.00.

The author believes that there are certain things which the average man should know and that these may be presented without the use of difficult mathematics. In his book, he has tried to explain the theoretical principles, operation, and management of electric generators, motors, etc., in a way which will be useful to those engaged in practical electrical work.

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* Unless otherwise specified, books in this list have been donated by the publishers.

PRACTICAL WIRELESS TELEGRAPHY:

A Complete Text Book for Students of Radio Communication. By Elmer E. Bucher. N. Y., Wireless Press, Inc. (copyright 1917). 322 pp., 323 illus., 9 x 6 in., cloth. $1.50.

After an exposition of those fundamentals of electricity which the telegraphist should know, the book describes the circuits of practical wireless sets and explains the basic principles of the apparatus associated with them. The author has chosen for description that apparatus which is most widely in commercial use.

STANDARD TABLE OF ELECTROCHEMICAL EQUIVALENTS

And Their Derivatives, with Explanatory Text on Electrochemical Calculations, Solutions of Typical Practical Examples and Introductory Notes on Electrochemistry. By Carl Hering and Frederick H. Getman. N. Y., D. Van Nostrand Co., 1917. 10 + 130 pp., 8 illus., 7 x 4 in., leather. $2.00.

Intended to provide a table of constants, based on the latest and best fundamental values, for calculating the amounts of substances electrolytically deposited, dissolved or otherwise chemically changed by an electric current.

THE ELECTRON:

Its Isolation and Measurement and the Determination of Some of Its Properties. By Robert Andrew Millikan. Chicago, The University of Chicago Press (copyright 1917). 12 + 261 pp., 33 illus., 8 x 5 in., cloth. $1.50.

Written to present the evidence for the atomic structure of electricity, to describe some of the most significant properties of the elementary electrical unit, and to discuss the bearing of these properties on the structure of the atom and the nature of electromagnetic radiation. Based on the author's researches in the Ryerson Laboratory, but includes a review of the work of former and contemporary investigators.

THE FLYING BOOK, 1917.

Edited by W. L. Wade. N. Y. and Lond., Longmans, Green and Co.; Lond., The Aviation World Publishing Co., Ltd., 1917. 291 pp., 165 illus., 6 pl., 9 x 6 in., boards. $1.25.

The subject matter is divided into three parts. Part 1 includes a number of brief articles on the design, control, and use of airplanes and engines; the present flight records; and alphabetical lists, with silhouettes and brief descriptions of engines, airplanes of to-day and airplanes of historic interest. Part 2 contains directories of aeronautical societies and of British, American, French, and Italian firms; and a biographical dictionary of aviators and manufacturers. Part 3, on military and naval aviation, includes the British honor list for 1916; a record of the work of the Zeppelin; and a diary of the events of 1916. A list of aeronautical publications, a glossary of terms, and various tables complete this handbook.

MODERN MACHINE SHOP:

Construction, Equipment, and Management; A Comprehensive and Practical Treatise on the Economical Building, the Efficient Equipment, and Successful Management of the Modern Machine Shop and Manufacturing Establishment. A Work for the Architect, Engineer, Manufacturer, Director, Officer, Accountant, Superintendent, and Foreman. By Oscar E. Perrigo. 2d ed., rev. and enl. N. Y., The Norman W. Henley Publishing Co., 1917. 384 pp., 219 illus., 9 x 6 in., cloth. $5.00.

Four new chapters have been added to this edition, treating of new developments in shop management. These deal with methods of increasing the efficiency of men and machines, with the relation of the overhead burden to the flat cost, and with manufacturing cost systems.

THE PRINCIPLES OF IRON FOUNDING.

By Richard Moldenke. N. Y., McGraw-Hill Book Co., Inc.; Lond., Hill Publishing Co., Ltd., 1917. 10 + 517 pp., 45 illus., 9 x 6 in., cloth. $4.00.

The principles underlying all the processes involved in the art of iron founding are given, together with an attempt to elucidate them, in the light of present metallurgical knowledge. Contents: Introduction; Industrial Status of the Foundry; Foundry Organization; Outline of Iron Metallurgy; Outline of Iron-Making Processes; Properties of Cast Iron; Classification of Castings; Foundry Raw Materials; The Technology of Combustion; Melting Processes; Mixture-Making; Testing of Cast Iron; Glossary of Foundry Terms; Appendix.

MILITARY PREPAREDNESS AND THE ENGINEER:

A Handbook for the Civilian Engineer. By Ernest F. Robinson. 2d ed. N. Y., The Clark Book Co., 1917. 244 + 361 pp., 132 illus., 7 x 4 in., leather. $2.00.

Written to place before the engineers of America as accurate an idea as possible of the opportunities and limitations that will confront the civilian engineer who is called to the colors, and to present the salient points of military engineering in the language with which he is familiar. This edition has been revised in the light of recent European experience, and considerable new matter has been added on field fortifications, wire entanglements, and the organization of captured positions.

WHAT A GEOLOGIST CAN DO IN WAR.

Prepared by R. A. F. Penrose, Jr., for the Geological Committee of the National Research Council. Philadelphia, J. B. Lippincott Co., 1917. 28 pp., 6 x 4 in., cloth. (Gift of the author.)

Suggests the services which the geologist can render in the selection of camp sites, ground for artillery positions, trench and tunnel locations, and other military operations.

THE IRON ORES OF LAKE SUPERIOR:

Containing Some Facts of Interest Relating to Mining and Shipping of the Ore and Location of Principal Mines. By Crowell and Murray. 3d ed., with Original Maps of the Ranges. Cleveland, The Penton Press Co., 1917. 316 pp., 7 illus., 13 maps, 9 x 6 in., cloth. (Gift of the authors.)

Contents: Early History of the Lake Superior Region; Geology; Mineralogy; Production of Ore; Dock Equipment; Classification of Lake Superior Iron Ores; Valuation of Lake Superior Iron Ores; Beneficiation of Ores; Methods of Analysis; Geology of the Wakefield Area of the Eastern Gogebic; Progress of Development of the Cuyuna Range; Description of Mines and Ores with Maps; Index to Mines and Ores.

UTILIZATION OF PYRITE OCCURRING IN ILLINOIS BITUMINOUS COAL.

By E. A. Holbrook. (University of Illinois *Bulletin*, Vol. 14, No. 51, August 20th, 1917.) Urbana, University of Illinois, 1917. 46 pp., 14 fig., 9 x 6 in., pap. 20 cents.

This pamphlet, which is *Circular No. 5* of the Engineering Experiment Station, is an outline of laboratory experiments performed on a commercial scale in the Mining Laboratory of the University with a view to developing a simple process for the economic recovery of the pyrite occurring with this coal and now commonly rejected as worthless. Plans of a suitable plant and estimates of its construction and operating costs are given, together with a description of the tests which led to its design. The author is convinced that the process is commercially feasible.

THE COMPOSITION OF TECHNICAL PAPERS.

By Homer Andrew Watt. N. Y., McGraw-Hill Book Co., Inc.; Lond., Hill Publishing Co., Ltd., 1917. 13 + 431 pp., 41 illus., 8 x 5 in., cloth. $1.50.

This book is a systematic text-book of English composition, and is intended to prepare the engineering student to write successfully such papers as his vocation will demand of him. It includes numerous examples of descriptive articles, expositions of processes and ideas, reports, and letters.

ORGANISATION PHYSIOLOGIQUE DU TRAVAIL.

Par Jules Amar. Préface de Henry Le Chatelier. Paris, H. Dunod et E. Pinat, 1917. 12 + 374 pp., 134 illus., 10 x 7 in., pap. 18 francs.

Written, the author states, "to assist the organization of labor according to national laws, the assignment to each of his true function in the social machine, the collaboration of the sound and the mutilated man in the work of to-morrow, to formulate in a few words the doctrine of the maximum utilization of the physical and psychical forces, without neglecting the moral factor." Describes the methods used in physiological laboratories to measure work and discusses the practical applications of the results in such systems as the Taylor system. One-third of the work discusses the re-education of the mutilated.

THE APPLICATION OF EFFICIENCY PRINCIPLES.

By George H. Shepard. N. Y., The Engineering Magazine Co., 1917. 10 + 368 pp., 7 diagrams, 9 x 6 in., cloth. $3.00.

Takes Harrington Emerson's statement of these principles and attempts to show how each can be practically applied. Illustrations from various fields of industry are given to show the operation of each principle. An endeavor has been made to strike a mean between a theoretical consideration of efficiency and a description of its application to one specific industry.

CORPORATE ORGANIZATION AND MANAGEMENT.

By Thomas Conyngton. Revised by H. Potter. N. Y., The Ronald Press Co., 1917. 26 + 778 pp., 9 x 6 in., cloth. $5.00.

A combination of the author's former books, "Corporate Management" and "Corporate Organization", in which duplicated material has been deleted and the remaining matter thoroughly modernized. Intended to furnish a convenient manual of corporation law and procedure for lawyers, corporation officials, and business men.

BUSINESS STATISTICS.

Edited by Melvin T. Copeland. Cambridge, Harvard University Press; Lond., Humphrey Milford, Oxford University Press, 1917. 12 + 696 pp., 3 diagrams, 9 x 6 in., cloth. $3.75.

A collection of articles, illustrating the methods of collecting, presenting, and using statistics relating to mercantile and manufacturing businesses, prepared primarily for purposes of instruction in the Harvard Graduate School of Business Administration. Contents: Statistical Methods; Statistical Indices of Business Conditions; Sales and Advertising Statistics; Factory Statistics; Statistics for the Chief Executive.

IS CIVILIZATION A DISEASE?

By Stanton Coit. Bost. and N. Y., Houghton-Mifflin Co.; The Riverside Press, Cambridge, 1917. 136 pp., 7 x 5 in., cloth. (Gift of the University of California Press.)

One of a series of essays on various phases of the moral law in its bearing on business life under the new economic order, delivered at the University of California and known as the Barbara Weinstock Lectures on the Morals of Trade.

BUSINESS LAW FOR ENGINEERS.

By C. Frank Allen. Part 1, Elements of Law for Engineers; Part II, Contract Letting. N. Y., McGraw-Hill Book Co., Inc.; Lond., Hill Publishing Co., Ltd., 1917. 431 pp., 9 x 6 in., cloth. $3.00.

The purpose of the book is to give the engineer a sufficient understanding of important fundamental features of law, so that he may have some idea of when or how to act himself and when to seek expert advice. The author has had experience as a civil engineer and has also practised law.

RESEARCHES OF THE DEPARTMENT OF TERRESTRIAL MAGNETISM.

Volume III: Ocean Magnetic Observations, 1905-1916; and Reports on Special Researches. By L. A. Bauer, with the Collaboration of W. J. Peters, J. A. Fleming, J. P. Ault, and W. F. G. Swann. Washington, Carnegie Institution of Washington, 1917. 447 pp., 11 x 9 in., pap. $10.00.

This volume presents the final results of the observations made aboard the *Galilee* and the *Carnegie* during the period designated. The special reports relate to atmospheric-electric observations, and to some discussions of the ocean magnetic work. The book is the third of the series of "Researches of the Department of Terrestrial Magnetism" published by the Carnegie Institution, the former two volumes of which treated of land magnetic observation.

OFFICIAL AUTOMOBILE BLUE BOOK 1917:

Standard Road Guide of America: Volume A, New York City and the Metropolitan District Embracing a Radius of 100 Miles from Columbus Circle. N. Y., Chicago, San Francisco, The Automobile Blue Book Publishing Co. (copyright 1917). 624 pp., illus., 1 map, 9 x 5 in., leather. $3.00.

The Blue Books cover the entire United States and Southern Canada in eleven volumes (see insert between pages 160 and 161). They tell tourists where to go and how to get there, giving complete maps of every motor road, running directions at every fork and turn, with mileages, all points of local or historical interest, State motor laws, hotel and garage accommodations, and ferry and steamship schedules and rates.

HISTORY OF TRANSPORTATION IN THE UNITED STATES BEFORE 1860.

Prepared, Under the Direction of Balthasar Henry Meyer, by Caroline E. MacGill and a Staff of Collaborators. Washington, Carnegie Institution, 1917. 11 + 678 pp., 6 maps, 10 x 7 in., pap. $6.50.

The Division of Transportation in the Department of Economics and Sociology of the Carnegie Institution of Washington presents this study of early American roads, railroads, and waterways as a contribution to American economic history, not as a completed study. An extensive bibliography is included, as well as colored maps showing the railroads in 1840, 1850, and 1860, present and abandoned canals, and the navigable rivers.

RAILROAD CONSTRUCTION:

Theory and Practice; A Text-Book for the Use of Students in Colleges and Technical Schools, and a Hand-Book for the Use of Engineers in Field and Office. By Walter Loring Webb. 6th. ed., rev. and enl. N. Y., John Wiley & Sons, Inc.; Lond., Chapman & Hall, Ltd., 1917. 15 + 831 pp., 218 illus., 7 x 4 in., cloth. $4.00.

The revision of the fifth edition has been so extensive that it has almost amounted to a rewriting of the book, comparatively few pages have not been changed. The work of the committees of the American Railway Engineering Association has been followed throughout, and considerable new matter on railroad surveys and the handling of surveying parties has been added.

SHAPE BOOK:

Containing Profiles, Tables, and Data Appertaining to the Shapes, Plates, Bars, Rails, and Track Accessories Manufactured by the Carnegie Steel Company. Pittsburgh (copyright 1917). 352 pp., 270 illus., 8 x 5 in., leather. $1.00.

TEXT-BOOK OF THE MATERIALS OF ENGINEERING.

By Herbert F. Moore. N. Y., McGraw-Hill Book Co., Inc.; Lond., Hill Publishing Co., Ltd., 1917. 10 + 204 pp., 69 illus., 1 pl., 9 x 6 in., cloth. $2.00.

The author presents herein a concise presentation of the physical properties of the common materials used in structures and machines, accompanied by brief descriptions of their manufacture and fabrication. The book is distinctly elementary in character. References to sources of additional information are included in each chapter.

PARALLEL TABLES OF SLOPES AND RISES:

In Combination with Diagrams of Slopes and Rises and Other Tables for Bridge and Structural Engineers, Draftsmen, Checkers, Templet Makers, Builders, and Vocational Schools. By Constantine K. Smoley. N. Y. and Lond., McGraw-Hill Book Co., 1917. 34 + 330 pp., 33 illus., 8 pl., 7 x 5 in., leather. $4.00.

Essentially an extension of Smoley's "Tables", this book consists of reprints from that volume, together with tables of slopes and rises. It differs from the former work, which is in the nature of a general calculator, in being concerned only with the solution of right triangles when one side and the bevel of the hypotenuse are known. It enables the calculator readily to determine the dimensions and bevels of the members of a structure, and also to lay out the joints and determine the dimensions of the material.

HANDBOOK OF CLEARING AND GRUBBING:

Methods and Cost. By Halbert Powers Gillette. N. Y., Clark Book Co., Inc., 1917. 240 pp., 67 illus., 7 x 5 in., cloth. $2.50.

This book contains a general review of all the methods, compiled from State and Federal bulletins, periodicals, and personal experience. Gives costs in detail whenever possible.

TIMBER FRAMING.

By Henry D. Dewell. San Francisco, Dewey Publishing Co., 1917. 275 pp., 112 illus., 9 x 6 in., cloth. $2.00. (Gift of the Mining and Scientific Press.)

The author presents the result of eleven years' experience in the design and superintendence of timber construction. The book is intended to supplement standard text-books on the subject by bringing into correlation drafting-room design and requirements of the field. The subject-matter was published originally in *Western Engineering*, but has been revised and enlarged.

EXAMINATION OF WATER:

Chemical and Bacteriological. By William P. Mason. 5th ed., rev. N. Y., John Wiley & Sons, Inc.; Lond., Chapman & Hall, Ltd., 1917. 186 pp., 9 illus., 7 pl., 8 x 5 in., cloth. $1.25.

The various methods given in former editions have been replaced by others more suited to modern practice, and the analytical methods have been harmonized with the recommendations of the 1917 Report of the American Public Health Association.

ENGINEERING FOR MASONRY DAMS.

By William Pitcher Creager. N. Y., John Wiley & Sons, Inc., Lond., Chapman & Hall, Ltd., 1917. 11 + 237 pp., 88 illus., 10 diagrams, 9 x 6 in., cloth. $2.50.

Contents: Investigations and Surveys; The Choice of Type of Dam; Forces Acting on Dams; Requirements for Stability of Gravity Dams; General Equations for Design of Gravity Dams; The Design of Solid Non-Overflow Gravity Dams; The Design of Solid Spillway Gravity Dams; The Design of Hollow Dams; The Design of Arch Dams; Preparation and Protection of the Foundation; Flood Flows; Details and Accessories.

THE ELEMENTS OF HYDROLOGY.

By Adolph F. Meyer. N. Y., John Wiley & Sons, Inc.; Lond., Chapman & Hall, Ltd., 1917. 12 + 487 pp., 287 illus., 2 diagrams, 3 maps, 9 x 6 in., cloth. $4.00.

Intended to provide engineers with a clear statement of the most important physical bases and applications of the fundamental principles underlying the science, rather than as a text or reference book. Contents: Introduction; The Atmosphere: Its Temperature, Pressure, and Circulation; Water: Its Various States and Their Properties; Precipitation: Its Occurrence and Distribution; Evaporation from Water Surfaces; Evaporation from Land Areas; Transpiration; Deep Seepage; Run-off: Stream-Flow Data; Supplementing Stream-Flow Data; Modification of Stream Flow by Storage; Notes to Teachers of Hydrology; Index.

MEMBERSHIP

(From September 7th to October 4th, 1917)

ADDITIONS

MEMBERS		Date of Membership.
ALLAN, FREDERICK WILLIAM. Gen. Supt., Empire Eng. Corporation, Inc., 1038 Marine Bank Bldg., Buffalo, N. Y.		Sept. 11, 1917
BASCOME, WESTERN RADFORD. Asst. to Chf. Engr., Dept. of Bridges, 41 Claremont Ave., New York City	Jun. Assoc. M. M.	Dec. 3, 1891 May 5, 1897 Sept. 11, 1917
BRANCH, LESTER VAN NOY. Res. Engr., U. S. Reclamation Service, Sherburne, Mont.	Assoc. M. M.	June 3, 1908 Sept. 11, 1917
CHRISTIE, CHESTER DE BAUN. Erecting Engr., Am. Bridge Co., 205 Wolvin Bldg, Duluth, Minn.		Sept. 11, 1917
CONNELL, WILLIAM HENRY. Eng. Executive, Day & Zimmerman, 611 Chestnut St., Philadelphia, Pa.	Assoc. M. M.	April 4, 1911 Sept. 11, 1917
DAILY, CORNELIUS MARK. Asst. Engr., St. Louis Water Co., 4240 Shaw Ave., St. Louis, Mo.		Sept. 11, 1917
DOELEMAN, HERMAN FRANÇOIS. Cons. Engr., 1101 American Bldg., Baltimore, Md.	Assoc. M. M.	June 24, 1914 Sept. 11, 1917
DRUMMOND, WILLIAM WORRELL. Civ. Engr. and Supt. of Constr. with U. S. War Dept., Fort H. G. Wright, N. Y.		Sept. 11, 1917
GANNETT, FARLEY. Cons. Engr., 204 Locust St., Harrisburg, Pa.	Assoc. M. M.	April 4, 1906 May 15, 1917
GEMBERLING, JOSEPH BURTON. Mgr., Eastern Div., Erecting Dept., Am. Bridge Co., 1712 Widener Bldg., Philadelphia, Pa.		Sept. 11, 1917
GLADDING, JAMES NICKERSON. City Engr., City Hall, El Paso, Tex.	Assoc. M. M.	Oct. 31, 1911 Sept. 11, 1917
GOUGH, WILLIAM JOSEPH. Chf. Engr. and Supt., Los Angeles & San Diego Beach Ry., 346 West Robinson Ave., San Diego, Cal.	Assoc. M. M.	Mar. 4, 1908 Sept. 11, 1917
GREENWOOD, ELMER ELLSWORTH. 279 Madison Ave., Skowhegan, Me.		May 15, 1917
HAWLEY, WILLIAM EVERETT. Engr., Duluth, Missabe & North. Ry., 401 Wolvin Bldg., Duluth, Minn.		Sept. 11, 1917
HAYDEN, ARTHUR GUNDERSON. Bridge Designer, New York State Barge Canal Office, 820 Park Ave., Albany, N. Y.		Sept. 11, 1917
HOOVER, ANDREW PEARSON. Associate, E. P. Goodrich, Cons. Engr., 261 Broadway, New York City	Assoc. M. M.	Mar. 4, 1914 Sept. 11, 1917

MEMBERS (*Continued*)

		Date of Membership.
HORNER, WESLEY WINANS. Engr. in Chg., Design of Sewerage and Paving, 325 City Hall, St. Louis, Mo.	Jun. Assoc. M. M.	Sept. 1, 1908 April 4, 1911 Sept. 11, 1917
HOUSER, SHALER CHARLES. Prof. of Eng., Univ. of Alabama, University, Ala.	Assoc. M. M.	Sept. 5, 1911 Sept. 11, 1917
JONES, WILLIAM NELSON. Chf. Engr., Filter Design and Constr., City Engr.'s Office, Minneapolis, Minn.	Assoc. M. M.	Oct. 31, 1911 Sept. 11, 1917
JOY, FREDERICK MERRICK. Chf. Engr., Land Dept., Tennessee Coal, Iron & R. R. Co., 1250 Brown Marx Bldg., Birmingham, Ala.		Mar. 13, 1917
KAUFFMANN, CHARLES EMIL. 46 South Prado, Atlanta, Ga.		Sept. 11, 1917
LA DU, DWIGHT B. Special Deputy State Engr., Barge Canal Office, Albany, N. Y.	Assoc. M. M.	Sept. 3, 1912 Sept. 11, 1917
LOCKWOOD, RICHARD JOHN. Vice-Pres. and Gen. Mgr., Apalachicola North. R. R., Port St. Joe, Fla.	Assoc. M. M.	May 3, 1910 Sept. 11, 1917
LYTEL, JAMES LEONARD. Project Mgr., U. S. Reclamation Service, Provo, Utah.	Assoc. M. M.	Jan. 4, 1905 Sept. 11, 1917
MCGLASHAN, HARRY DEYOE. Dist. Engr., U. S. Geological Survey, 328 Custom House, San Francisco, Cal.	Assoc. M. M.	Oct. 31, 1911 Sept. 11, 1917
MANSFIELD, NEWTON. Lt.-Commander, U. S. Navy (*Retired*), U. S. Naval Recruiting Station, Anchor Bank Bldg., Pittsburgh, Pa.		Sept. 11, 1917
MARTIN, LEWIS M. Dist. Engr., Iowa State Highway Comm., Atlantic, Iowa.	Assoc. M. M.	Oct. 7, 1914 Sept. 11, 1917
MERRILL, ROBERT HALL. Asst. Engr., Barge Canal Office (Res., 70 North Allen St.), Albany, N. Y.	Jun. Assoc. M. M.	May 3, 1904 May 28, 1912 Sept. 11 1917
PIERCE, CHARLES HENRY. Dist. Engr., U. S. Geological Survey, Custom House Bldg., Boston, Mass.	Assoc. M. M.	April 30, 1912 Sept. 11, 1917
POWELL, HERBERT JAMES BINGHAM. 75 Belsize Park Gardens, London, N. W., England.		Sept. 11, 1917
RANDOLPH, RICHARD WOOD. Engr.-in-Chf., I-Kwei Section, Szechuen-Hankow Ry., Ichang, Hueph, China.	Assoc. M. M.	Dec. 6, 1910 June 12, 1917
ROBERTS, THEODORE CHARLES. Hotel Cumberland, New York City.		Sept. 11, 1917
SHIKLES, JAMES WALLACE. Field Engr., Div. of Valuation, Interstate Commerce Comm., 510 North 5th Ave., La Grange, Ill.		Sept. 11, 1917
SISSOEFF, SERGEI NICOLAEVITCH. 1 Matweewskaja St., Petrograd, Russia.		Mar. 13, 1917

MEMBERS (*Continued*)

		Date of Membership.
SYRETT, WALTER SUTTON. Pres., Walter S. Syrett & Co., 72 West Adams St., Chicago, Ill.		Sept. 11, 1917
TIBBETTS, FREDERICK HORACE. (Haviland, Dozier & Tibbetts), 1320 Alaska Commercial Bldg., San Francisco, Cal.	Jun.	May 1, 1906
	Assoc. M.	April 6, 1909
	M.	Sept. 11, 1917

ASSOCIATE MEMBERS

ASPLUNDH, EDWIN THEODORE. Capt., 103d Reg., Engrs., 28th Div., Camp Hancock, Augusta, Ga.		Sept. 11, 1917
BARKER, CHARLES ROBERT. Engr., National Board of Fire Underwriters, 174 Auburn St., Auburndale, Mass.		Sept. 11, 1917
BASCOM, GEORGE ROCKWELL. Asst. Prof., Municipal and San. Eng., Extension Div., Univ. of Wisconsin, 1819 University Ave., Madison, Wis.		Sept. 11, 1917
BAUGHMAN, CHARLES ALTON. Instr., Civ. Eng., Iowa State Coll., 622 West 7th St., Ames, Iowa		Sept. 11, 1917
BAYLISS, PAUL. Asst. Engr., Public Service Comm., Clarence, Mo.		Sept. 11, 1917
BEARD, ROBERT STANLEY. Asst. City Engr., Kansas City, Mo.		Sept. 11, 1917
BEAVEN, EDUARDO. Engr., The National Bank of Mexico; Engr., Cia de Bienes Muebles e Inmuebles, S. A., 8ª Flores 146, City of Mexico, Mexico		Sept. 11, 1917
BERKEFELD, JOHN WOLFRAM. 227 Bonita Ave., Piedmont, Cal.	Jun.	April 7, 1915
	Assoc. M.	Sept. 11, 1917
BLOSSER, EMMET CHANDLER. Div. Engr., State Highway Dept., 2342 Indianola Ave., Columbus, Ohio		Sept. 11, 1917
BLUHM, HERMAN WILLIAM. 328 Guion Ave., Richmond Hill, N. Y.	Jun.	Nov. 8, 1909
	Assoc. M.	Sept. 11, 1917
CAFFALL, GEOFFREY ARTHUR. 1143 East 15th St., Brooklyn, N. Y.	Jun.	Sept. 3, 1912
	Assoc. M.	Sept. 11, 1917
CHIPLEY, DUDLEY. Engr. and Supt., Water-Works, P. O. Box 854, Columbus, Ga.		Sept. 11, 1917
CROSBY, LOTHROP. Asst. Engr., Idaho Irrig. Co., Richfield, Idaho		Sept. 11, 1917
CUNLIFF, NELSON. Commr., Parks and Recreation, 330 Municipal Courts Bldg., St. Louis, Mo.		Sept. 11, 1917
DRINKWATER, ERNEST. Municipal Engr., 588 Desaulniers Boulevard, St. Lambert, Que., Canada		Sept. 11, 1917
FINLEY, GUY CEPHAS. Supt. of Irrig., U. S. Reclamation Service, Naches, Wash.		Sept. 11, 1917
FLEMING, JOSEPH HAMILTON. Asst. County Surv., Franklin County, 77 East Lane Ave., Columbus, Ohio		Sept. 11, 1917
FORTER, CECIL ALFRED. 13 The Devon, Topeka, Kans.		Sept. 11, 1917
FUNK, NORMAN WILL. Junior Structural Engr., Interstate Commerce Comm., 601 Interstate Bldg., Kansas City, Mo.		Sept. 11, 1917

ASSOCIATE MEMBERS (*Continued*)

		Date of Membership.
HASKINS, JOHN CHRISTOPHER. Chf. Engr., Louisville Bridge & Iron Co., Louisville, Ky.	Jun. Assoc. M.	May 28, 1912 Sept. 11, 1917
HUNT, NELSON BARNES. Chf. Draftsman, Office U. S. Surveyor-General, 103 Capital Apartments, Olympia, Wash.		Sept. 11, 1917
HUSTED, ALVA GUY. Asst. Engr., Subdivision of Sewage Disposal, Div. of Eng., 5811 Thackeray Ave., Cleveland, Ohio		Sept. 11, 1917
McDONNELL, FRANCIS REGIS. Care, Revere Sugar Refinery Co., Medford St., Charlestown, Mass.		Sept. 11, 1917
McMULLEN, RAY WEBB. Engr., Arthur McMullen, 50 Vanderbilt Ave., New York City	Jun. Assoc. M.	June 30, 1910 Sept. 11, 1917
MOFFETT, JOHN WILLIAM. Chf. Engr., The Sperry Eng. Co., 105 Dwight St., New Haven, Conn.		Sept. 11, 1917
MOREY, EDWARD FRANCIS. P. O. Box 334, Dallas, Tex.		Sept. 11, 1917
MORRISON, ROGER LEROY. Prof. of Highway Eng., Agricultural and Mechanical Coll. of Tex., College Station, Tex.	Jun. Assoc. Assoc. M.	April 4, 1911 Mar. 2, 1915 Sept. 11, 1917
MOSS, JEROME AARON. Traveling Supt., Jos. E. Nelson & Sons, 118 North La Salle St., Chicago, Ill.		Sept. 11, 1917
MURDOCH, RUSSELL ALGER. Cons. Engr., 706 Free Press Bldg., Detroit, Mich.		Sept. 11, 1917
MURPHY, ALVIN RUSH. Fountain City, Tenn.	Jun. Assoc. M.	June 6, 1911 Sept. 11, 1917
NANCE, ARCHIBALD WHITFIELD. Secy. and Treas., Farris Eng. Co., Empire Bldg., Pittsburgh, Pa.		Sept. 11, 1917
NORTON, JOHN EVERED. Asst. Field Engr., Interstate Commerce Comm., 731 Wells Fargo Bldg., San Francisco, Cal.		Sept. 11, 1917
PORTER, ARTHUR MILLER. Los Banos, Cal.		Mar. 13, 1917
QUINN, THOMAS FRANCIS. Asst. Engr., Trans. Miss. Terminal R. R., Pierce City, Mo.		Sept. 11, 1917
RAYMOND, HERBERT DWIGHT. Andersonville, Ga.		Sept. 11, 1917
REES, BIRD LE GRAND. Cresskill, N. J.		Sept. 11, 1917
ROBINSON, RUSSELL MOORE. 803 Columbia St., Hudson, N. Y.	Jun. Assoc. M.	Mar. 2, 1915 Sept. 11, 1917
ROSENBAUM, RAYMOND VICTOR. Care, Potomac Light & Power Co., Martinsburg, W. Va.		Sept. 11, 1917
RUDOLPH, WILLIAM EDWARD. Asst. Engr., Public Service Comm., First Dist. (Res., 399 Hancock St.), Brooklyn, N. Y.	Jun. Assoc. M.	Oct. 1, 1912 Sept. 11, 1917
SCHULTZ, HAROLD AUGUST HASTRUP. 2624 Second Ave., Detroit, Mich.	Jun. Assoc. M.	Mar. 2, 1915 May 15, 1917

ASSOCIATE MEMBERS (*Continued*)

		Date of Membership.
SEIB, CHARLES BACH. Maintenance Engr., New York State Highway Comm., Box 815, Kingston, N. Y.	Jun. Assoc. M.	Sept. 2, 1914 April 17, 1917
SHEPARD, GEORGE MILSON. Capt., Engrs., U. S. R., Fort Snelling, Minn.		June 11, 1917
SLAYMAKER, CHARLES MONROE. Div. Engr., State Highway Dept., 511 Metropolitan Bldg., East St. Louis, Ill.		April 17, 1917
SMITH, WALTER GROVER. Div. Engr., Ohio State Highway Dept., 2295 Adams Ave., Columbus, Ohio		Sept. 11, 1917
SPRAGUE, HUGH MAX. Asst. City Engr., Box 445, Great Falls, Mont.		Sept. 11, 1917
SWEM, ANZI RAYMOND. Asst. Chf. Engr., Iowa Ry. & Light Co., Box 115, Kenwood Park, Iowa		April 17, 1917
TIDD, HARRY. 110 East 13th St., Hutchinson, Kans.	Jun. Assoc. M.	Mar. 14, 1916 Sept. 11, 1917
TOLMAN, EDWARD MAYO. Director and Chf. Engr., Div. of San. Eng., West Virginia State Dept. of Health, Box 1346 Charleston, W. Va.	Jun. Assoc. M.	Sept. 2, 1914 Sept. 11, 1917
VOGEL, JOSHUA HOLMES. Archt., Engr., and Vice-Pres., W. M. Vories & Co., Hachiman, Omi, Japan		Sept. 11, 1917
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Holborn, Lewis Adams. 2d Battery, F. A., R. O. T. C., Military Branch, Chattanooga, Tenn.	Sept. 11, 1917
Jenks, Harry Neville. San. Engr., Burma Mines, Ltd., Namtu, Northern Shan States, Burma, India.	May 15, 1917
McClung, David Arthur. Hotel Tioga, San Diego, Cal.	Sept. 11, 1917
Warwick, Charles Laurence. 209 Merion Ave., Narberth, Pa.	June 11, 1917

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SPEAKMAN, RICHARD EDWARD. Elected Member, January 4th, 1910; died January 13th, 1917.

Total Membership of the Society, October 4th, 1917, 8 469.

MONTHLY LIST OF RECENT ENGINEERING ARTICLES OF INTEREST

It has been necessary to omit this list from this number of PROCEEDINGS *due to conditions incident to preparation for the removal of Society Headquarters from Fifty-seventh Street to Thirty-ninth Street. Its publication will be resumed in the November issue.*

—SECRETARY.

MECHANICAL DRAFTSMEN
A PATRIOTIC OPPORTUNITY

Here is another opportunity for patriotic men to serve their country. The Ordnance Department needs mechanical draftsmen, and, unless the situation is corrected, war preparations will be seriously retarded. The eyes of the Nation are chiefly centered on its military strength, but the need for educated, trained men for this indispensable civilian work behind the lines is of equal importance.

If the civilian force is efficient, the Army is promptly and adequately supplied and it fights a winning battle. If there is inefficiency and delay, the Army is hampered, the war is prolonged, life is sacrificed needlessly, and taxes are increased. The errors of our Allies in the early part of the war were directly traceable to administrative inefficiency.

Owing to the demand of private business, there is great difficulty in obtaining mechanical draftsmen for the Government, and it is the duty of every properly equipped American to offer his services.

Red tape has been dispensed with. It does not mean long routine with examinations requiring the assembling of candidates. It means simply an intelligent investigation of the fitness of every volunteer and an immediate rating of such volunteer according to ability. In practice this takes the form of requiring from volunteers a full statement of education and experience, and, on the basis of these statements, together with the recommendations of reputable persons, the appointments are made without delay.

At the request of the Ordnance Department, the **National Civil Service Reform League**, at 79 Wall Street, New York City, is answering inquiries concerning these positions, and will furnish application forms on request.

PAPERS AND DISCUSSIONS

OCTOBER, 1917

Vol. XLIII. OCTOBER, 1917. No. 8.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

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AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

This Society is not responsible for any statement made or opinion expressed in its publications.

PULSATIONS IN PIPE LINES, AS SHOWN BY SOME RECENT TESTS

By H. C. Vensano, M. Am. Soc. C. E.

To be Presented November 7th, 1917.

Synopsis.

The writer, while recently occupying the position of Civil and Hydraulic Engineer of the Pacific Gas and Electric Company, had the good fortune to be enabled to make certain experiments and measurements of pulsations (water-hammer) in long pipe lines, and these are described in this paper. This information, it is believed, will prove a valuable addition to the present literature on the subject. The experiments should be of particular value because they were made on a line under actual operating conditions. They show what can be expected, practically, in the way of wave effects, and demonstrate that pulsations, whether due to gate opening or closing, can by no means be neglected in design, even for lines which are controlled by slowly moving gates.

The Drum Power Plant, of the Pacific Gas and Electric Company, put into service in 1913, is a hydro-electric generating station on the Bear River, in Placer County, California, in which there are

Note.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and, when finally closed, the papers, with discussion in full, will be published in *Transactions*.

two 12 500-kw. electric generators, each driven by a pair of Pelton impulse wheels. The supply for these wheels is brought to the power-house in a riveted steel pipe, or penstock, 6 282 ft. long, and having a diameter of 72 in. at its upper end and 52 in. at the power-house. The experiments discussed herein were made on this penstock. Pulsations of acceleration due to gate opening and also of retardation due to gate closing were investigated. These effects were obtained by opening or closing (simultaneously) one or more of the needle nozzles which control the supply to the wheels. Owing to the fact that the time of closure for a nozzle was approximately 69 sec., the results will apply to the conditions of slow gate closure. Inasmuch as most of the experiments in the past (as far as the writer knows) have been on rapid gate closures (so rapid as to produce as nearly as possible an instantaneous condition), and inasmuch as the conditions usually occurring in practice deal with slow closure, the following results should be of considerable value in assisting the engineer to ascertain the water-hammer to be expected in penstock lines under practical conditions of design and operation.

A study of the curves obtained from the experiments described herein has led the writer to the following conclusions:

First.—That the general formula for pressure variation from normal at any point in a pipe line, with uniformly varying gate opening, should be

$$h = \frac{2\,(L_1\,V_1 + L_2\,V_2 + \;.\;.\;.\;.\;.\; L_{x-1}\,V_{x-1} + L_x\,V_x)}{g\,T} \ldots (1)$$

This formula applies to a pipe with varying diameter.

Second.—For slow gate closure, this formula reduces to

$$h = \frac{2\,(L_1\,V_1 + L_2\,V_2 + L_3\,V_3 +, \text{etc. for the full length of the pipe})}{g\,T} \ldots (2)$$

and, further, reduces to

$$h = \frac{2\,L_1\,V_1}{g\,T} \ldots\ldots\ldots\ldots (3)$$

for slow gate closure with a pipe of uniform diameter, as advocated by Professor Joukovsky. Formulas (1), (2), and (3) are limited to a maximum value of

$$h = \frac{a_1\,v_1}{g} \ldots\ldots\ldots\ldots (4)$$

Third.—That, as this formula indicates, the velocity of flow at the gate, or at the point where the pressure is to be ascertained, does not necessarily (of itself alone) fix the magnitude of the pressure wave at such point, but that the magnitude of the pressure wave is influenced by the varying velocities of the moving water column in all portions of the line between the point at which the pressure is to be ascertained and the reservoir.

Fourth.—That, under ordinary conditions, the water-hammer effect may, and does, produce as great a fall in pressure below the normal as it produces a rise above normal after the gate has been closed completely. In other words, the pressure vibrates back and forth above and below normal after gate closure.

Fifth.—That, in pipes of uniform diameter, the magnitude of pressure variation along the pipe line will vary directly as the time required for the wave to travel from any point in question to the reservoir and return to the same point, as advanced by Professor Joukovsky, provided the time of gate closure is greater than the half period of the pipe.

Sixth.—That the effect of accelerating the water column by gate opening is analogous to the effect of retardation in gate closing, except that the pressure variations have the opposite sign. The period of pulsation is the same. The chief difference is that the wave effects die out much more rapidly with opening than with closing, and this seems also to damp the vibration so rapidly that the full magnitude is obtained only for a short time.

Seventh.—That the synthetic method, used by Professor Joukovsky and Miss Simin to determine wave forms, can be used to good advantage in the study of such effects, and can be made to predict probable wave forms and magnitudes, if properly interpreted

Eighth.—It is here found that the velocity of wave propagation in water, for riveted steel pipe, can be calculated approximately by the recognized formula:

$$a = \frac{12}{\sqrt{\frac{W}{g}\left(\frac{1}{K} + \frac{D}{E\,b}\right)}} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (5)$$

if proper allowance is made for the effect of joint details.

General Description of Experiments.

In order to avoid making this paper too long, the writer assumes that the reader is familiar with certain literature on the subject already available. In fact, before examining this paper, he should read at least two papers to which the writer refers, particularly "Water-Hammer", by Miss O. Simin*, and also that part of a paper entitled "Penstock and Surge-Tank Problems",† by Minton M. Warren, Assoc. M. Am. Soc. C. E., applying to pressure due to slow or ordinary gate closures, together with (and especially) the discussion on that paper by the writer. Although it will be found that the writer (due to the results of the experiments described herein) has modified in some particulars the views he expressed in that discussion, certain fundamental ideas were brought out there at considerable length, and will not be repeated herein.

The experiments were made on the penstock of the Drum Power-House, Fig. 1. The gate arrangement at the lower end of the pipe line consists of four needle nozzles controlling an equal number of Pelton impulse wheels. The arrangement of the nozzles is shown in Fig. 5. The nozzles are operated electrically, and tests were first made to determine the rate, period, and characteristics of their opening and closing. They were specially designed to give a uniform rate of gate opening, but practically do not quite do so. The opening and closing curves obtained are shown by Fig. 6, which gives the relation of area of opening to time of gate motion. The opening curve shows an almost perfectly uniform rate of opening. The closing curve shows a somewhat slow start, probably due to lag in the motor operating the gate, and a somewhat accelerated finish, perhaps due to the increasing water pressure behind the needle of the nozzle as the flow was checked. Each experimental point on the diagram is the result of a number of tests.

During the tests the nozzles—which are deflecting nozzles—were directed away from the wheels. It is well to note here that the arrangement is such that, for the full open position of either one or more nozzles, the velocity in the 26-in. pipe, to which Gauge No. 1 was attached, remains practically constant.

* *Proceedings,* American Water Works Assoc., 1904, p. 341. This is a review and elaboration of the article by Professor J. Joukovsky in *Memoirs,* Imperial Academy of Sciences, St. Petersburg, 1897, Vol. IX.

† *Transactions,* Am. Soc. C. E., Vol. LXXIX, p. 238.

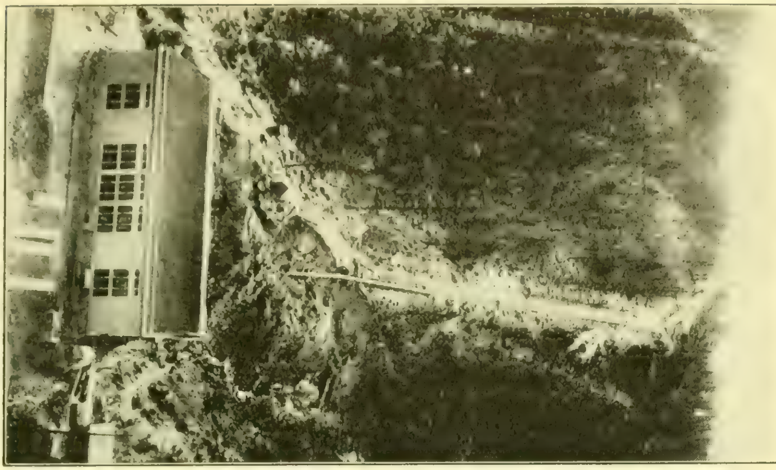


FIG. 1.—DRUM POWER-HOUSE, AND LOWER END OF PENSTOCK.

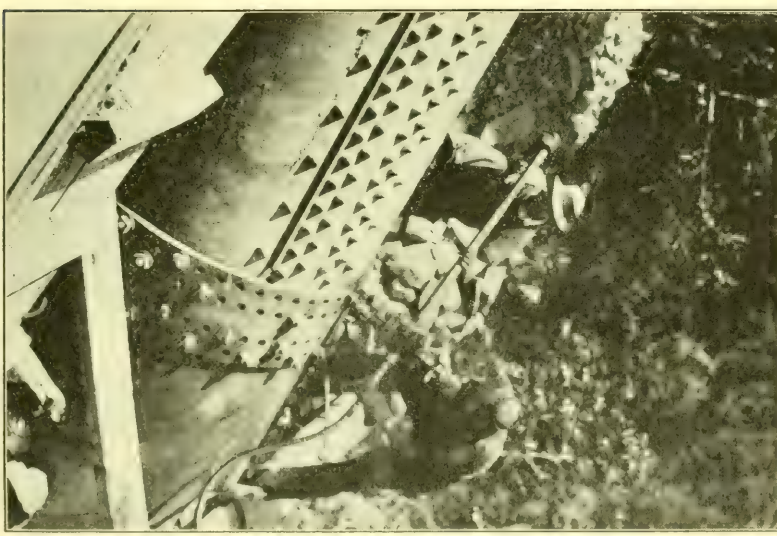


FIG. 2.—DETAIL OF PENSTOCK, DURING CONSTRUCTION.

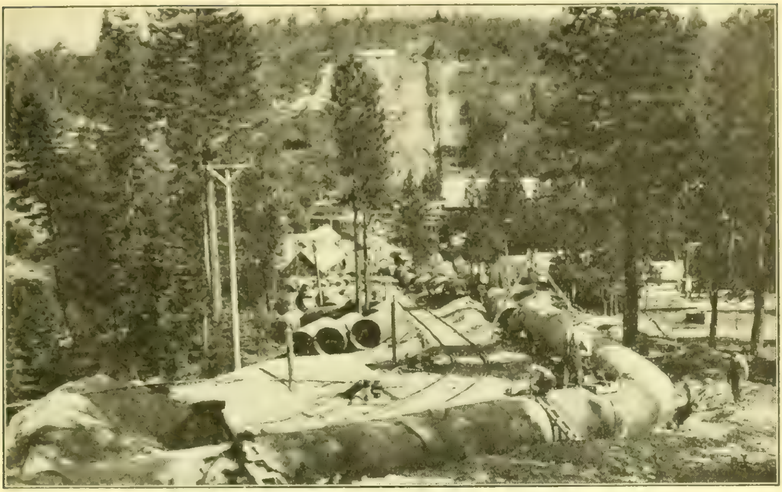


FIG. 3.—DRUM PENSTOCK, 72 INCHES IN DIAMETER.

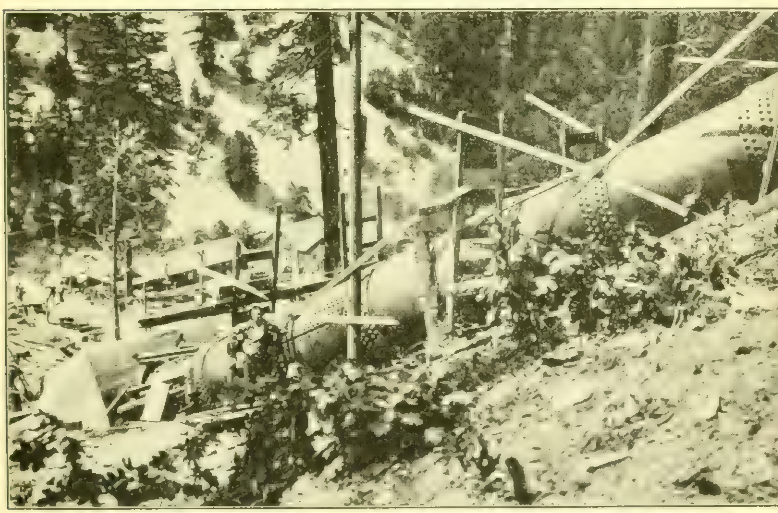
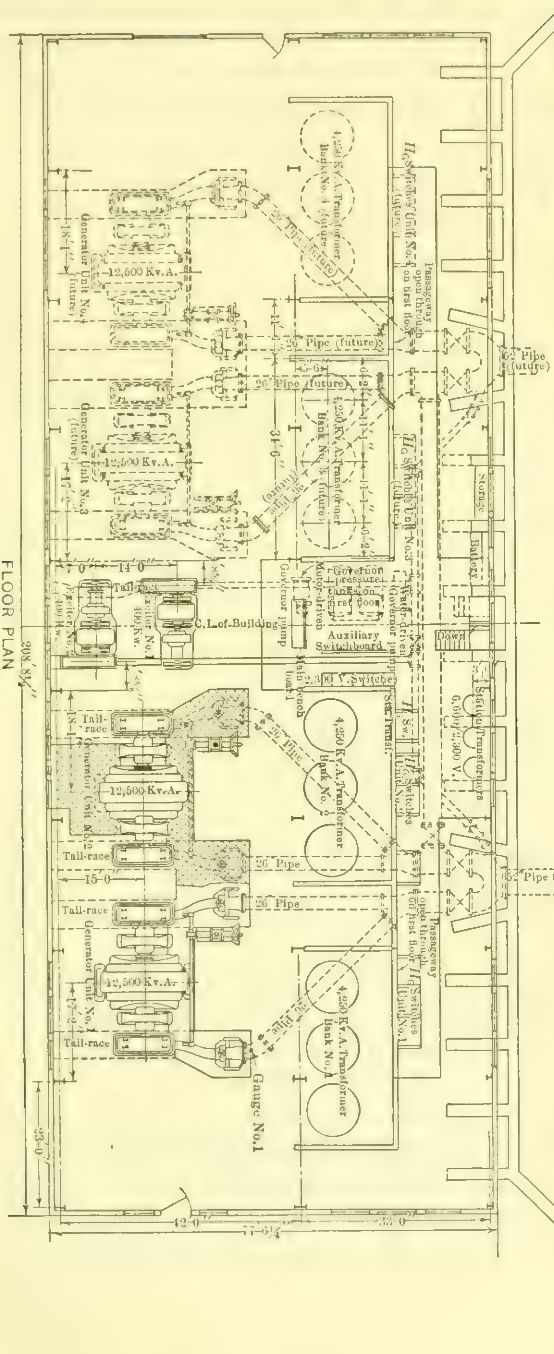


FIG. 4.—DRUM PENSTOCK, 72 INCHES IN DIAMETER.



GENERAL ARRANGEMENT OF DRUM POWER HOUSE

FLOOR PLAN

FIG. 5.

Gauges.—The gauges used were Bourdon recording gauges, with the exception of No. 1, which was a Bourdon indicating gauge, and each had a range of pressure selected for the point at which it was to be used. All gauges were tested and calibrated carefully imme-

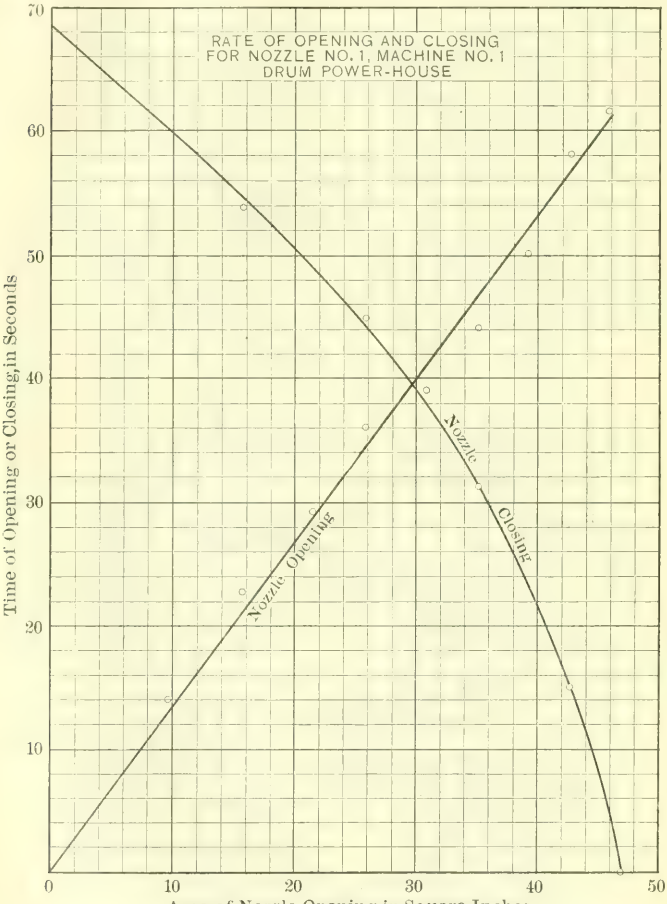


FIG. 6.

diately before being used. Gauge No. 1, at the power-house, being indicating only, was read for minimum and maximum points.

In considering the resulting curves, it is to be noted that the readings at the power-house gauge (No. 1) and gauge charts at two other points were taken simultaneously. This gives comparative

results for effects of the same waves at points varying in distance from the gate.

Nomenclature.—

A = Area of cross-section of full gate opening, in square feet;

A_1, A_2, A_3, A_4, etc. = Area of cross-section of pipe, in square feet, corresponding to the diameter of D_1, D_2, D_3, D_4;

a = Velocity of wave propagation in a pipe of uniform diameter;

a_1, a_2, a_3, a_4, = Velocity of wave propagation for lengths of pipe, L_1, L_2, L_3, L_4, etc.;

b = Thickness of pipe walls, in inches;

D = Diameter of pipe, in inches, where line is of constant diameter;

D_1, D_2, D_3, D_4, etc. = Diameter of pipe corresponding to lengths, L_1, L_2, L_3, L_4, in inches;

E = Coefficient of elasticity of steel taken at 30 000 000 lb. per sq. in.;

g = Acceleration due to gravity, in feet per second = 32.2;

h = Pressure rise, or fall from normal, measured in feet of head at any point (due to pulsations);

k = Voluminal modulus of elasticity of water, taken at 294 000 lb. per sq. in.;

L = Total length of pipe, in feet, between the point at which the pressure is desired and the reservoir;

L_1, L_2, L_3, L_4, etc. = Length of sections of pipe of varying diameter, in feet, between the point at which the pressure is desired and the reservoir, L_1 being the first section, starting at the point at which the pressure is desired;

l_x = A partial length of section, L_x, such that

$$\frac{2\,l_x}{a_x} = T - \left[\frac{2\,L_1}{a_1} + \frac{2\,L_2}{a_2} + \;.\;.\;.\;.\; \frac{2\,L_{x-1}}{a_{x-1}}\right];$$

P = Pressure, in pounds, due to head, h;

T = Time of gate closure (or opening), in seconds;

t = Time, in seconds, for wave to travel from any point to reservoir and return to that point;

t_0 = One-half period of pipe $= \frac{2\ L}{a}$ for pipe of uniform diameter, $= \frac{2\ L_1}{a_1} + \frac{2\ L_2}{a_2} + \frac{2\ L_3}{a_3}$, etc., for pipe of varying diameter, taking L_1 from the gate and including all sections to the reservoir;

V = Velocity of flow in pipe of uniform diameter, in feet per second, at beginning of gate motion;

V_1, V_2, V_3, V_4, etc. = Velocity of flow, in feet per second, in sections of pipe of diameter, D_1, D_2, D_3, D_4, respectively, at beginning of gate motion;

v_1, v_2, v_3, v_4, etc. = Velocity of flow, in feet per second, in sections of pipe of diameter, D_1, D_2, D_3, D_4, respectively, at end of gate motion;

w = Weight of water per cubic foot, taken at 62.4 lb.;

V_x, a_x, A_x, are values corresponding to V_1, a_1, A_1, at the point, X, in the pipe line.

Slow gate closure is defined as one in a time greater than t_0. Rapid gate closure is one in a time less than t_0.

Experiments.—The following experiments for various gate motions were made:

Test No. 1.

July 15th, 1915.

Four nozzles opened simultaneously, left open 5 min., and closed simultaneously. Record taken at Gauge Points Nos. 2 and 3.

Test No. 2.

July 15th, 1915.

Four nozzles opened simultaneously, left open 5 min., and closed simultaneously. Records taken at Gauge Points Nos. 2 and 3.

Test No. 3.

July 15th, 1915.

Four nozzles opened simultaneously, left open 5 min., and closed simultaneously.

Chart No. 3-A (Fig. 7) Gauge No. 2. Both opening and closing.

Chart No. 3-B (Fig. 8) Gauge No. 3. Both opening and closing.

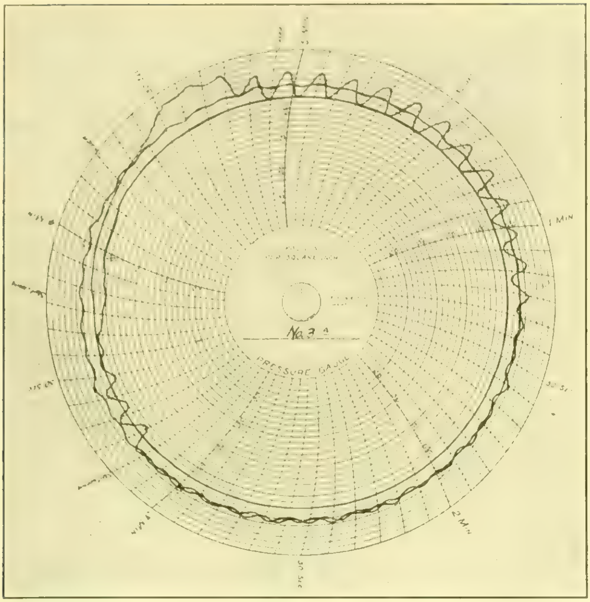


Fig. 7.

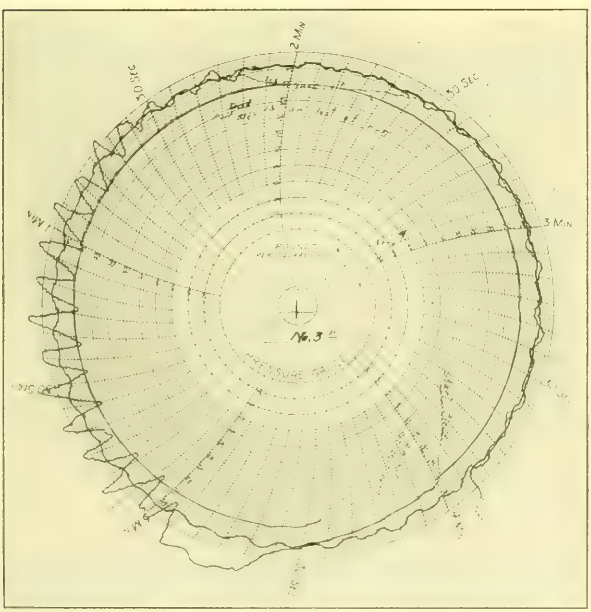


Fig. 8.

Test No. 6.

July 19th, 1915.

Two nozzles opened simultaneously, left open 3 min., and closed simultaneously. Records taken at Gauge Points Nos. 1 and 2.

Readings on Gauge No. 1.

581½ lb. closed	566 closing	585 6 min.
570 ¾ min.	568	578
571	572	585
560	580 5½ min.	584
564	595	584
562	597	580 8½ min.
564 open	574	
	584	
	574	
	590	

Note.—The time indicated opposite these pressures is from the time of starting to open.

Test No. 7.

July 19th, 1915.

Two nozzles opened simultaneously, left open 3 min., and then closed simultaneously. Records taken at Gauge Points Nos. 1 and 2.

Readings on Gauge No. 1.

581½ lb. closed	670	684
76	74	82
70	84	584
58	95	80
62	97	83
63	85 closed	80
64 open	78	83
665 closing	79	

Test No. 8.

July 19th, 1915.

Two nozzles opened simultaneously, left open 3 min., and then closed simultaneously. Records taken at Gauge Points Nos. 1 and 2.

Readings on Gauge No. 1.

581½ lb. closed	566 closing	574	584	582	
74	68	82	80	81	
72	70	88	84	81	7¼ min. total
62	74	78	83		
58	90	86	84		
64 open	98	78	83		
	96 closed	84	80		

Test No. 9.

July 19th, 1915.

Two nozzles opened simultaneously, left open 3 min., and closed simultaneously.

Chart No. 9-B (Fig. 9) Gauge No. 2. Closing nozzles.
Chart No. 9-D (Fig. 10) Gauge No. 3. Closing nozzles.

Readings on Gauge No. 1.

581½ lb. closed	568 closing	578	582	580	
74	72	86	80	84	8¼ min. total
72	80	82	82		
66	96	79	78		
58	96	86	84		
62	80 closed	80	80		
64 open	88	78	84		

Test No. 10.

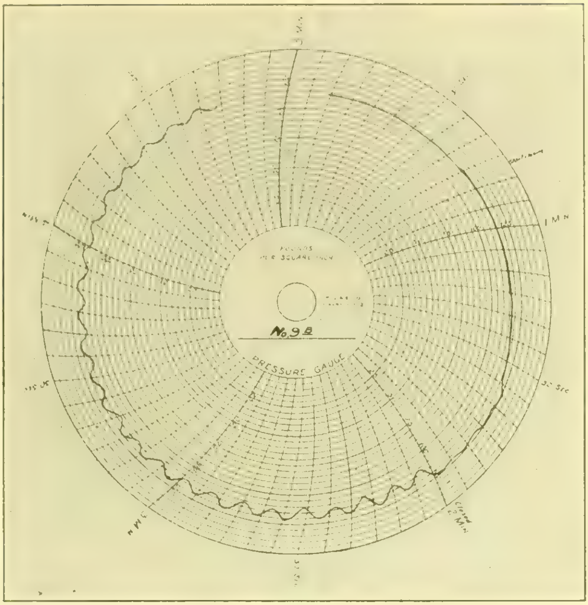
July 19th, 1915.

Two nozzles opened simultaneously, left open 3 min., and then closed simultaneously.

Chart No. 10-A (Fig. 11) Gauge No. 2. Opening nozzles.
Chart No. 10-C (Fig. 12) Gauge No. 3. Opening nozzles.

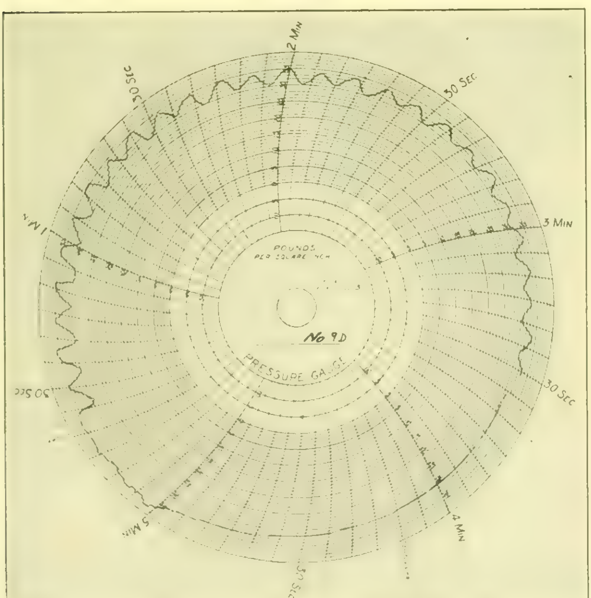
Readings on Gauge No. 1.

581½ lb. closed	567	569	594	583	
72	558	72	96 closed	81	
76	564 open	80	80	82	
71	566 closing	98	84	82	Total time 7 min.



No. 9 B
PRESSURE GAUGE
1 MIN
30 SEC

FIG. 9.



2 MIN
30 SEC
3 MIN
30 SEC
4 MIN
30 SEC
5 MIN
30 SEC
1 MIN
30 SEC
POUNDS
PER SQUARE INCH
No 9D
PRESSURE GAGE

FIG. 10.

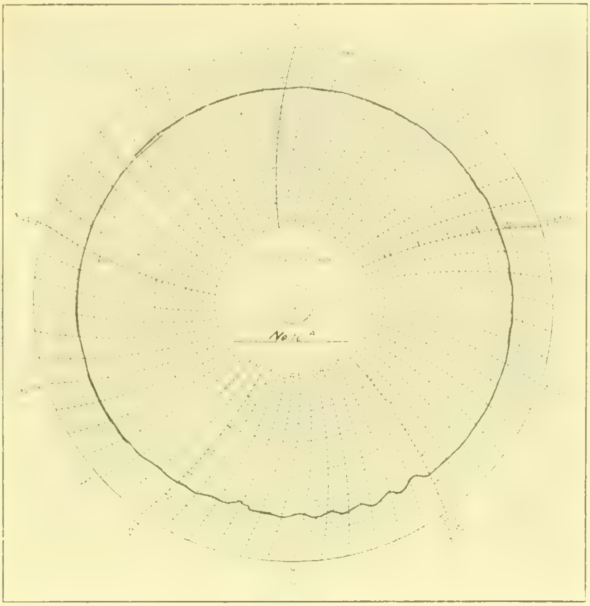


FIG. 11.

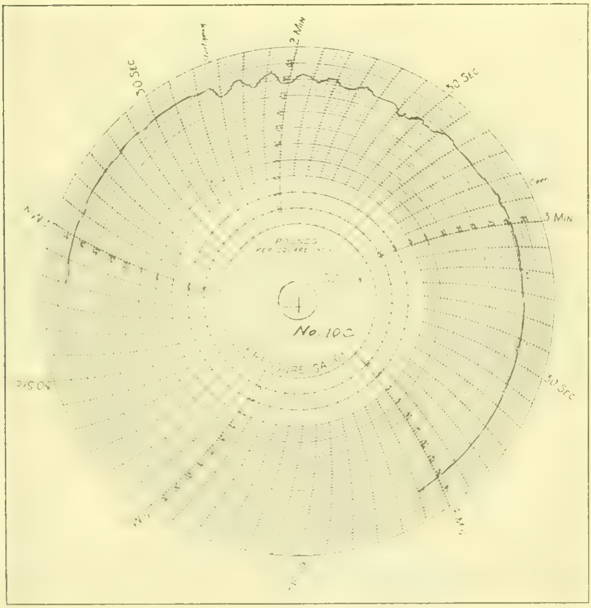


FIG. 12.

Test No. 13.

July 19th, 1915.

Opened one nozzle, left open 3 min., and then closed. Records taken at Gauge Points Nos. 1, 2, and 3.

Readings on Gauge No. 1.

581½ closed	92 closing
70	84 total time 7 min. 40 sec.

Test No. 14.

July 19th, 1915.

Opened one nozzle, left open 3 min., and then closed.

Chart No. 14-A (Fig. 13) Gauge No. 2. Opening nozzle.
" No. 14-B (Fig. 14) Gauge No. 2. Closing nozzle.
" No. 14-C (Fig. 15) Gauge No. 3. Opening nozzle.
" No. 14-D (Fig. 16) Gauge No. 3. Closing nozzle.

Readings on Gauge No. 1.

581½ closed	576 closing
80	78
78	88
70	92
72	90
74 opened	82
	84

Test No. 15.

July 19th, 1915.

Four nozzles opened simultaneously, held open 3 min., and then closed simultaneously.

Chart No. 15-A (Fig. 17) Gauge No. 2. Opening nozzles.
Chart No. 15-B (Fig. 18) Gauge No. 2. Closing nozzles.
Chart No. 15-C (Fig. 19) Gauge No. 3. Opening nozzles.
Chart No. 15-D (Fig. 20) Gauge No. 3. Closing nozzles.

Gauge Readings.

582 lb. closed	540 open	598
80	48 closing	570–98 vibrating
63	56	74–92 "
58	70	78–90 "
44	600	76–92 total time 7¾ min.
32	608	

Test No. 16.

July 19th, 1915.

Four nozzles opened simultaneously, held open 3 min., and then closed simultaneously. Records taken at Gauge Points Nos. 1, 2, and 3.

Chart No. 16-A (Fig. 21) Gauge No. 2. Opening nozzles.

Chart No. 16-B (Fig. 22) Gauge No. 2. Closing nozzles.

Readings on Gauge No. 1.

582½ lb. closed	534	556	610
70	38	70	575
62	40 open	608	80
58	48 closing	612	85 total time 7 min.

Test No. 18.

July 18th, 1915.

Opened two nozzles simultaneously, held open 3 min., and then closed simultaneously. Records taken at Gauge Points Nos. 1, 3, and 4.

Readings on Gauge No. 1.

582 closed	75	562	578	595
72	70	65	80	98 closed
78	70	66 open	84	
74	62	70 closing	88	Readings taken
80	61	72	97	about every ¼
72	65	74	98	min.
76	65	76	96	

Note.—Evidently had a different man reading this gauge than for the previous experiments.

Test No. 19.

July 18th, 1915.

Opened two nozzles simultaneously, held open 3 min., and then closed simultaneously. Records taken at Gauge Points Nos. 1, 3, and 4.

Chart No. 19-A (Fig. 23) Gauge No. 4. Opening nozzles.

Chart No. 19-B (Fig. 24) Gauge No. 4. Closing nozzles.

Readings on Gauge No. 1.

582 closed	573 closing
570	599
560	582
566 open	583 closed. Readings taken every ½ min.

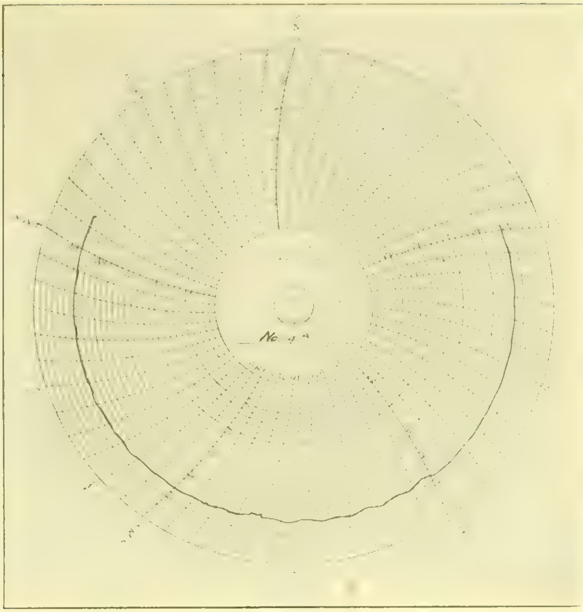


FIG. 13.

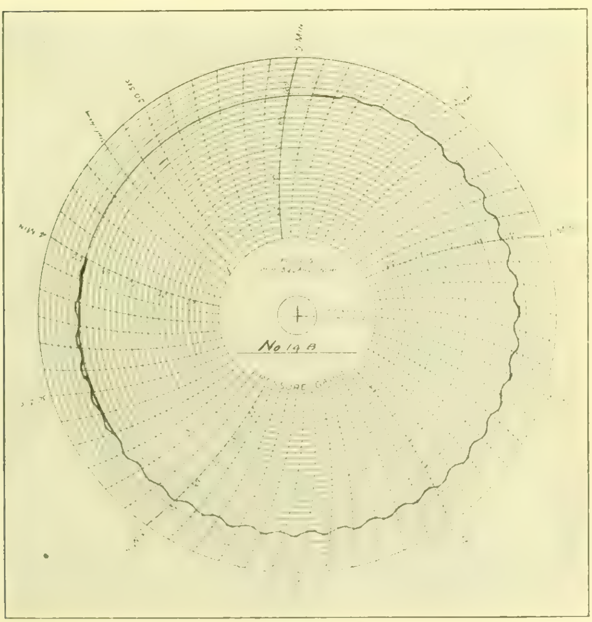


FIG. 14.

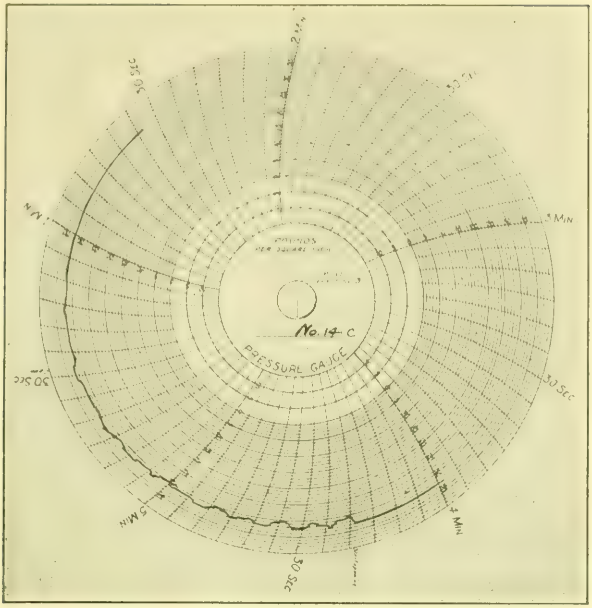

2 MIN
30 SEC
3 MIN
No. 14 C
POUNDS PER SQUARE INCH
PRESSURE GAUGE
30 SEC
4 MIN
30 SEC
5 MIN
30 SEC

FIG. 15.

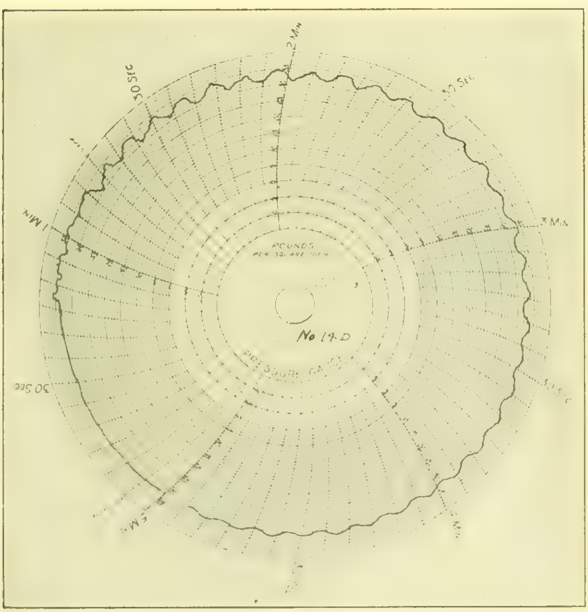

2 MIN
30 SEC
30 SEC
1 MIN
3 MIN
POUNDS PER SQUARE INCH
No 14 D
30 SEC
30 SEC

FIG. 16.

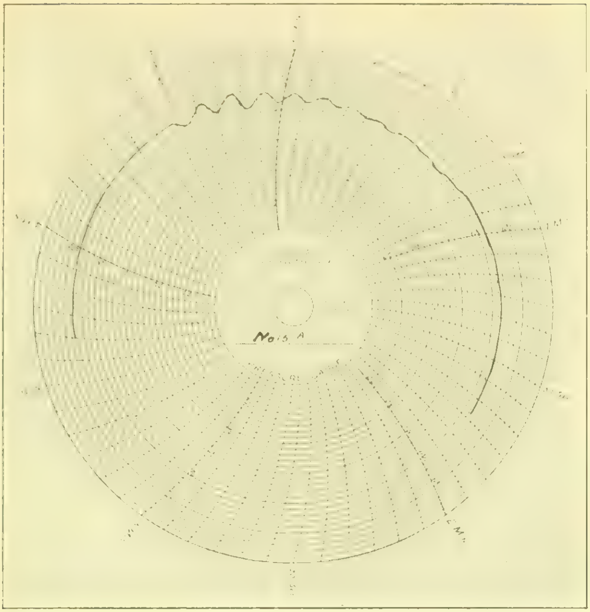


Fig. 17.

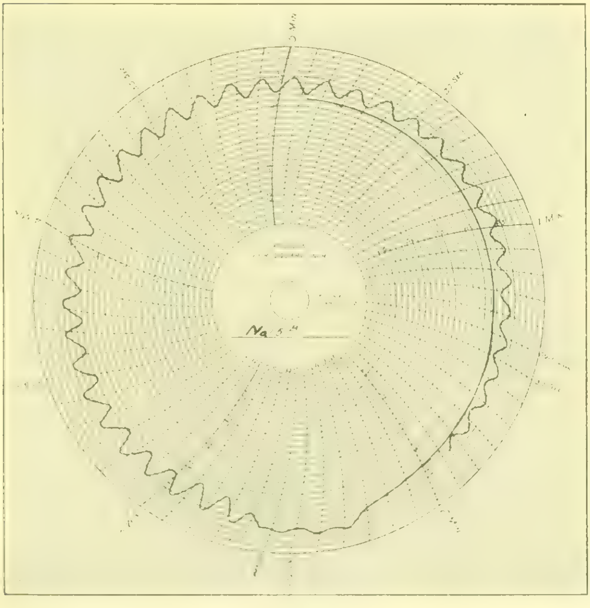


Fig. 18.

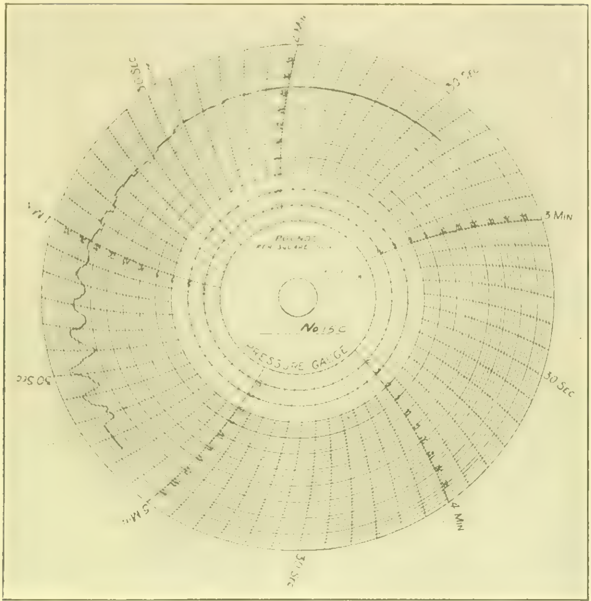

3 MIN
30 SEC
4 MIN
POUNDS PER SQUARE INCH
No. 15 C
PRESSURE GAUGE

FIG. 19.

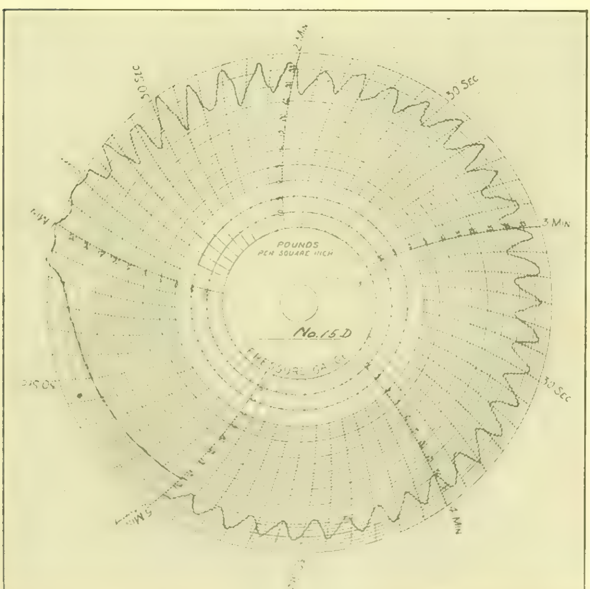

30 SEC
3 MIN
POUNDS PER SQUARE INCH
No. 15 D
PRESSURE GAUGE
4 MIN

FIG. 20.

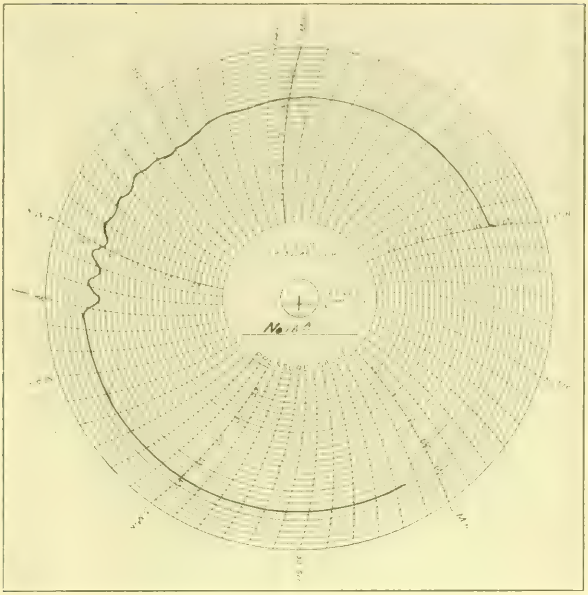


FIG. 21.

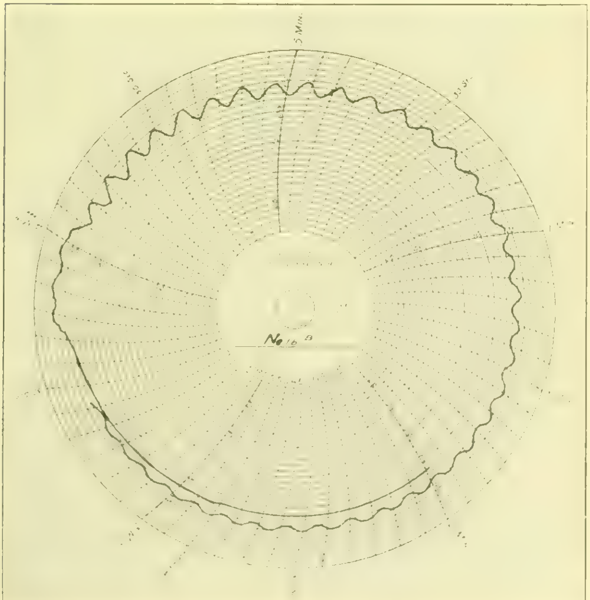


FIG. 22.

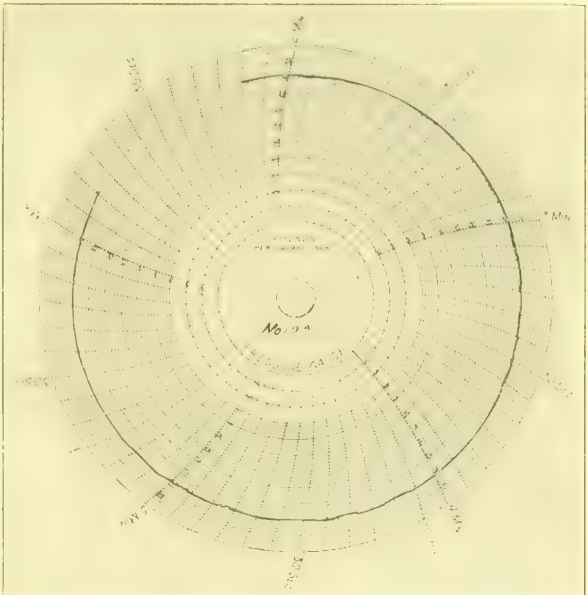

30 Sec
No. 19 A
30 Sec

FIG. 23.

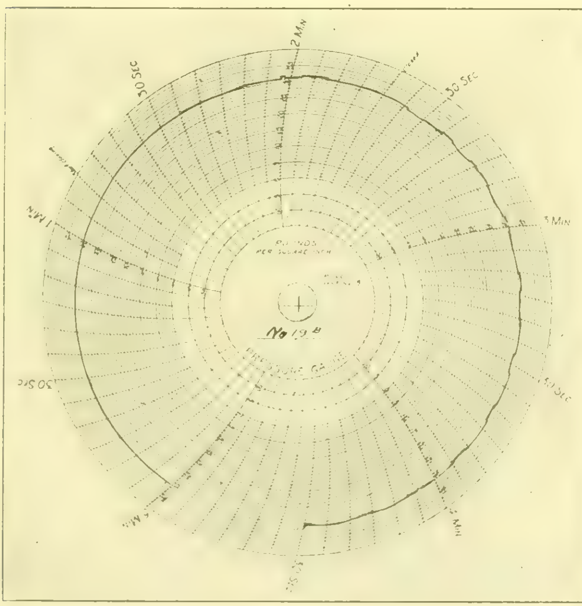

2 Min
30 Sec
30 Sec
1 Min
3 Min
No. 19 B
30 Sec
30 Sec

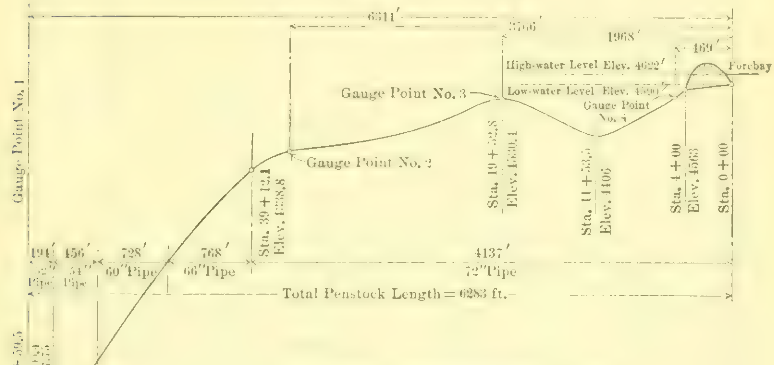
FIG. 24.

It is to be noted that charts were obtained for each and every test, but, in the foregoing, only the numbers of those charts which are published herein are given. These, however, are typical of the others, and, in fact, the similarity of the charts produced by a repetition of the same gate motion is remarkable.

A summary of the results of all the tests is given in Table 1.

Velocity.—The quantity of water flowing has been ascertained from the pressure readings at the nozzle by using the formula,

$$Q = A \sqrt{\frac{2\,g\,H}{\left(\frac{1}{C}\right)^2 - \left(\frac{A}{A_1}\right)^2}} \qquad \text{................(6)}$$



PROFILE OF DRUM PENSTOCK LINE
SHOWING POSITION OF GAUGES

FIG. 25.

and, from this value, the velocities at various points of the pipe line have been deduced.

Where A = area of the nozzle opening = 46.96 sq in. = 0.326 sq. ft.;

A_1 = area of the pipe leading to the nozzle = 405.4 sq. in. = 2.82 sq. ft.;

C = the coefficient of discharge = 0.96;

H = the pressure head at the entrance to the nozzle, as shown by Gauge No. 1.

Pipe Line.—Table 2 states the physical data of the pipe line.

The nozzle arrangement and the position of Gauge No. 1 are shown on Fig. 5. This line was of riveted steel.

TABLE 1—(*Continued*).

Experiment No.	Gauge No.	L, in feet.	Q, in second-feet.	V, at gauge.	P, static.	P, flow.	OPENING (OBSERVED). P, minimum.	OPENING (OBSERVED). Time, in seconds.	OPENING (OBSERVED). P, normal.	p.	CLOSING (OBSERVED). P, maximum.	CLOSING (OBSERVED). Time, in seconds.	CLOSING (OBSERVED). P, normal.	p.	Calculated values of p, in pounds.
13	2	3 766	94	3.32	117.5	116	112	41	117.1	5.1	122.5	53	117.4	5.1	
14	......	3 766	......	......	117.5	116.5	111.5	40	117.0	5.5	122	52	117.3	4.7	
Average	......	......	......	......	......	......	......	......	......	5.3	......	......	......	4.9	4.87
1	3	1 968	365	12.9	36.3	30.1	24	40	34.3	10.3	47	54	36.0	11.0	
2	......	......	......	......	36.4	30	24.9	40	34.4	9.5	46	50	35.9	10.1	
3	......	......	......	......	36.3	30	24.2	39	34.3	10.1	47	50	35.8	11.2	
15	......	......	......	......	32.8	26.8	23	45	30.3	7.3	41.5	53	32.4	9.1	
16	......	......	......	......	32.5	27	22	42	30.5	8.5	42	58	32.3	9.7	
Average	......	......	......	......	......	......	......	......	......	9.2	......	......	......	10.2	9.9
9	3	1 968	186	6.6	32.6	31	27.2	43	32.0	4.8	37.9	52	32.5	5.4	
10	......	......	......	......	32.6	30.5	26.5	43	31.8	5.3	38.8	50	32.5	6.3	
Average	......	......	......	......	......	......	......	......	......	5.1	......	......	......	5.8	5.6
13	3	1 968	94	3.32	32.4	32.1	29	41	32.3	3.3	36	53	32.4	3.6	
14	......	......	......	......	32.7	32.1	29	40	32.5	3.7	36	52	32.7	3.3	
Average	......	......	......	......	......	......	......	......	......	3.5	......	......	......	3.4	2.54
18	4	469	186	6.6	30.7	29.9	29	43	30.4	1.4	31.9	53	1.2	......	
19	......	......	......	......	30.7	30.0	29.1	43	30.4	1.4	31.9	48	1.2	......	
Average	......	......	......	......	......	......	......	......	......	1.4	......	......	1.2	......	1.2

Calculated wave period = 8.76 sec.
Actual wave period = 6.95 sec.

TABLE 1.—PRESSURE VARIATIONS IN DRUM PENSTOCK DUE TO OPENING AND CLOSING OF NEEDLE NOZZLES.

Experiment No.	Gauge No.	L, in feet.	Q, in second-feet.	V, at gauge.	P, static.	P, flow.	Opening (Observed). P, minimum.	Time, in seconds.	P, normal.	p.	Closing (Observed). P, maximum.	Time, in seconds.	P, normal.	p.	Calculated values of p, in pounds.
15	1	6 311	365	32.4	582	540	532	52	558	26	608	54	580	28	
16					582.5	540	534	45	564	30	612	54	580.5	32.5	
Average										28				30.25	37
6	1	6 311	186	33	581.5	564	560	40	575.6	15.6	597	45	579.1	17.6	
7					581.5	564	558	43	574.7	16.7	597	40	578.5	18.5	
8					581.5	564	558	43	574.7	16.7	598	38	578.0	20.0	
9					581.5	564	558	43	574.7	16.7	596	52	580.5	15.5	
10					581.5	564	558	45	574.2	16.2	598	48	579.7	18.3	
18					582	566	561	43	575.8	14.8	598	40	579.2	18.8	
19					582	566	560	42	576.0	16.0	599	48	580.4	18.6	
Average										16.1				18.2	19.05
13	1	6 311	94	33.4	581.5	573	570	41	578.5	8.5	592	53	581.0	11.0	
14					581.5	574	570	40	579.0	9	592	52	581.0	11.0	
Average										8.75				11.0	
1	2	3 766	365	12.9	120	109	99	41	116.2	17.2	140	52	119.3	20.7	
2					120.5	109	99	40	116.7	17.7	140	47	119.4	20.6	
3					121.0	109.5	100	39	117.2	17.2	142	53	120.5	21.5	
15					117.5	107	103	45	113.1	10.1	135	54	117.0	18.0	
16					117.5	107	99	42	113.8	14.7	136	60	117.8	18.7	
Average										15.4				19.9	18.9
6	2	3 766	186	6.6	116	113	107.5	40	115	7.5	126	45	115.6	10.4	
7					117	113.5	105	43	115.7	10.7	127.5	40	116.4	11.1	
8					116	113	105	43	114.9	9.9	126	38	115.4	10.6	
9					116	113	107	43	114.9	7.9	126	52	115.8	10.2	
10					116.5	113.5	105.5	45	115.8	9.8	126	48	116.2	9.8	
Average										9.2				10.4	9.72

TABLE 2.—PHYSICAL DATA OF PIPE LINE.

Type of pipe.	Diameter, in inches.	Thickness of metal, in inches.	Length, in feet.
Lap-riveted.	72	$\frac{1}{4}$	1 714.6
" "	72	$\frac{5}{16}$	1 147
" "	72	$\frac{3}{8}$	480
Butt strap.	72	$\frac{3}{8}$	730
" "	72	$\frac{7}{16}$	65
" "	66	$\frac{7}{16}$	244
" "	66	$\frac{1}{2}$	123
" "	66	$\frac{9}{16}$	119
" "	66	$\frac{5}{8}$	103
" "	66	$\frac{11}{16}$	97
" "	66	$\frac{3}{4}$	82
" "	60	$\frac{3}{4}$	114
" "	60	$\frac{13}{16}$	153
" "	60	$\frac{7}{8}$	96
" "	60	$\frac{15}{16}$	98
" "	60	1	71
" "	60	$1\frac{1}{16}$	97
" "	60	$1\frac{1}{8}$	99
" "	54	$1\frac{1}{16}$	91
" "	54	$1\frac{1}{8}$	113
" "	54	$1\frac{3}{16}$	117
" "	54	$1\frac{1}{4}$	134.4
" "	52	$1\frac{1}{4}$	194.17
Cast steel.	36	$1\frac{1}{2}$	18
" "	26	$\frac{3}{4}$	33.25 aver.
" " taper.	26 to 20	$1\frac{1}{8}$	4
		Total length......	6 337.42

Fig. 25 is a diagrammatic profile of the line showing the positions of the gauges used to record the pulsation effects. The end of the pipe line at the forebay reservoir is wide open.

Magnitude of Pulsation.—A study of these charts developed the fact that the magnitude of the pulsation (that is, the wave pressure) was more nearly in accord with Professor Joukovsky's formula $\left(\frac{2\,L\,V}{T\,g}\right)$ than with some of those advanced more recently. It was observed, however, that although there was a rather close agreement for Gauge Points Nos. 2, 3, and 4, the formula did not accord at all with the results at Point No. 1. As Gauge Points Nos. 2, 3, and 4, were in such positions that the pipe above them was of uniform diameter, and the pipe above Gauge No. 1 was of varying diameter, it was decided to endeavor to obtain a formula applicable to a line of varying diameter.

By a study of the principles advanced by Professor Joukovsky, Formula 1 was developed for a pipe of varying diameter. It will be remembered that, after proving that the maximum possible wave pressure due to instantaneous gate closure was $\frac{a\,V}{g}$, Professor Joukovsky

assumed that a gate closure in finite time could be taken as made up of a succeeding series of partial instantaneous gate closures, in which case, the pressure would rise by increments at the rate of

$$\frac{\frac{a\,V}{g}}{T},$$

and that this pressure rise would continue for the time, T, or until a reflected wave from the reservoir would cut it short in a time, $\frac{2\,L}{a}$.

Assuming such to be the case for a pipe of uniform diameter, it must be evident that, in a pipe of varying diameter, any section of the line of larger diameter may be regarded as an imaginary reservoir, or a point of pressure relief for all points below it, or rather between it and the gates. For, although an instantaneous gate motion will produce a wave, $\frac{a_1\,V_1}{g}$, in magnitude, in a pipe of diameter, D_1, it will continue on through a pipe of diameter, D_2, with a magnitude only of $\frac{a_2\,V_2}{g}$.* Therefore, when a wave reaches the first point of increase in diameter in the line, a partial relief of pressure occurs, and a return relief wave is started back toward the gate, where it will arrive in the time, $\frac{2\,L_1}{a_1}$. Similarly, partial relief waves will be started at each point of change in the section of the pipe. Following this conception, it is evident that a pressure wave started at the gate will increase at the rate, $\frac{\frac{a_1\,V_1}{g}}{T}$, for a time, $\frac{2\,L_1}{a_1}$, when the first partial relief wave reaches the gate. Then, the pressure will continue to increase at the rate, $\frac{\frac{a_2\,V_2}{g}}{T}$, for a time, $\frac{2\,L_2}{a_2}$, etc., until, during a final period equal to,

$$\left\{ T - \left(\frac{2\,L_1}{a_1} + \frac{2\,L_2}{a_2} + \frac{2\,L_3}{a_3} + \quad . \quad . \quad . \quad . \quad + \frac{2\,L_{x-1}}{a_{x-1}} \right) \right\}$$

the rise is at the rate of $\frac{\frac{a_x\,V_x}{g}}{T}$. When the gate completes its motion, the

* See Professor Joukovsky's method of calculation of maximum pressure in paper by O. Simin.

maximum pressure at the gate is reached. This maximum pressure will then evidently be,

$$h = \frac{a_1\,V_1}{g\,T} \cdot \frac{2\,L_1}{a_1} + \frac{a_2\,V_2}{g\,T_1} \cdot \frac{2\,L_2}{a_2} + \quad . \quad . \quad . \quad . \quad + \frac{a_{x-1}\,V_{x-1}}{g\,T} \cdot \frac{2\,L_{x-1}}{a_{x-1}}$$

$$+ \frac{a_x\,V_x}{g\,T}\left(T - \left\{\frac{2\,L_1}{a_1} + \frac{2\,L_2}{a_2} + \quad . \quad . \quad . \quad . \quad + \frac{2\,L_{x-1}}{a_{x-1}}\right\}\right) \,..(7)$$

Now, let,

$$T - \left(\frac{2\,L_1}{a_1} + \frac{2\,L_2}{a_2} + \quad . \quad . \quad . \quad . \quad + \frac{2\,L_{x-1}}{a_{x-1}}\right) = \frac{2\,l}{a_x} \,......(8),$$

producing a convention whereby, for the purpose of calculation, the pipe line is terminated at the last point from which an imaginary relief wave could return to the gate, arriving in just a time, T, when the gate completes its motion.

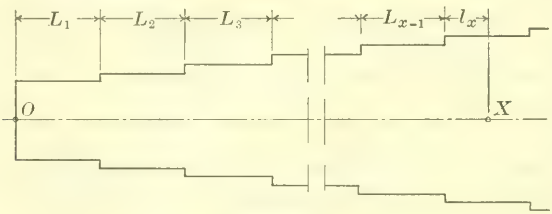


FIG. 26.

Introducing this value for T in Equation (7) reduces it to the form

$$h = \frac{2}{g\,T}\,(L_1\,V_1 + L_2\,V_2 + \quad . \quad . \quad . \quad . \quad + L_{x-1}\,V_{x-1} + l_x\,V_x) \,..(1)$$

Having solved Equation (8), $L_1 + L_2 + L_3 + L_{x-1} + l_x$ will be the length of line to consider in fixing maximum pressure, and may be represented by OX in Fig. 26, where O represents the gate and X the farthest point from it. Note that, ordinarily, X will not be a point of change of pipe section, and that, ordinarily, l_x will not be a full length of section.

Referring to Equation (1), it is evident that, for slow gate closure, as T is less than t_0, X falls in the reservoir and V_x is 0, and the equation becomes

$$h = \frac{2\,(L_1\,V_1 + L_2\,V_2 + L_3\,V_3 + \quad . \quad . \quad . \quad \text{etc.})}{g\,T} \,.........(2)$$

in which all sections of the pipe are used for the full length of the

line. This is, then, the equation for slow gate closure with pipe of varying diameter.

For a pipe of uniform diameter with slow gate closure, Equation (2) reduces to the form,

$$h = \frac{2\,L_1\,V_1}{g\,T} \qquad \text{(3)}$$

the formula advanced by Professor Joukovsky.

Although these equations were deduced for pressure at the gate, it is evident that they will hold true for any point in the line, inasmuch as the first wave produced by gate closure at any point in the line is the same as though an imaginary gate were closed at this point in the same time, T, as the actual gate is closed. In reference to this, one should note carefully the definition of L_1, L_2, L_3, etc., as these vary for different points.

In Equations (1), (2), and (3), h is limited to a maximum value equal to $\frac{a_1\,V_1}{g}$, and this maximum will occur at some point in the line for all values of $T < \frac{2\,L_1}{a_1}$ and for all points in a line of uniform diameter with instantaneous gate closure.

In order not to lengthen this paper unduly, the writer will not discuss this limiting value and its application here, but will refer to his discussion of Mr. Warren's paper (previously mentioned) in which he has gone rather fully into the matter.

Equation (2) is the one applicable to the experiment described herein because of the slow gate closure. Equation (2) may also be deduced mathematically, as follows, using as a basis the proposition of mechanics that

Impulse = Force $\times$ Time = Change in Momentum.

The total change in momentum,

$$= \left[\frac{w\,A_1\,V_1\,L_1}{g} - \frac{w\,A_1\,v_1\,L_1}{g}\right] + \left[\frac{w\,A_2\,L_2\,V_2}{g} - \frac{w\,A_2\,v_2\,L_2}{g}\right] + \text{etc.} \qquad \text{(9)}$$

or, where the change in momentum is measured from or to a complete state of rest, this becomes

$$\frac{W\,A_1\,L_1\,V_1}{g} + \frac{W\,A_2\,L_2\,V_2}{g} + \text{etc.} \qquad \text{(10)}$$

as $v_1 = v_2 = v_3 = 0$.

The impulse, or product of force and time, would be $\frac{W\,A\,h}{2} \times T$, where A represents the area of the gate opening and $\frac{h}{2}$ is used as the average pressure head.

The assumption that the average force during the time of gate movement is equal to the maximum force divided by two would seem to be logical, in that, as the gate closes uniformly, it exposes to the flowing column a surface of impedance which varies uniformly from zero to a maximum. It would seem, therefore, that the force resisting the flow would vary in the same ratio.

Equating, we have,

$$\frac{W\,A\,h}{2}\,T = \frac{W\,A_1\,L_1\,V_1}{g} + \frac{W\,A_2\,L_2\,V_2}{g} + \text{etc.} \ldots\ldots (11)$$

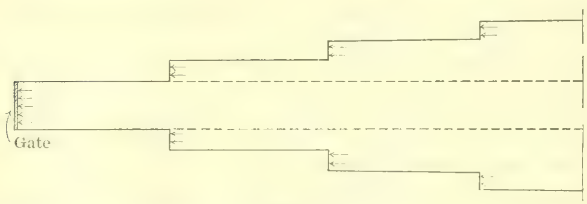


FIG. 27.

It is to be noted, however, that, although the cross-sectional area of the pipe may vary, only that portion of the water column having a cross-sectional area equal to that of the gate opening will be stopped directly by the pressure developed on the gate. (See Fig. 27.) The remainder of the resisting force will be developed at the points of change of diameter of the pipe by the increased pressure at these points.

This formula, therefore, becomes Equation (2)

$$h = \frac{2\,(L_1\,V_1 + L_2\,V_2 + L_3\,V_3 + \quad . \quad . \quad . \quad . \quad \text{etc.})}{g\,T} \ldots\ldots (2)$$

It might be noted that Mr. Warren, in using the impulse momentum principle to develop a pressure formula for slow gate closure, has assumed the time element for the impulse action to be equal to $\left(T + \frac{L}{a}\right)$, in place of T, in view of the fact that the water column is not completely brought to rest until a time, $\frac{L}{a}$, after the gate has finally completed its motion. This, however, the writer believes to be incorrect,

for the following reason: The first impulse increment or wave at the beginning of gate motion, although immediately affecting the water at the gate, would not affect the final portion of the water column at the reservoir until a time, $\frac{L}{a}$, after the gate had begun its motion. Similarly (as Mr. Warren points out), the effect of the last increment of force from the gate would not affect the portion of the water column at the reservoir until a time, $\frac{L}{a}$, after the gate had completed its motion, but, evidently, these two effects balance, and every portion of the water column has been acted on only for the time, T.

Normal Pressure Curve.—In interpreting the charts, it must be noted carefully that the value of h here referred to is the increased pressure due to pulsations only, and, therefore, measurements of h must be made with reference to the normal position of the hydraulic gradient at the same instant. As the gate moves, there is a gradual change in the normal position of the hydraulic gradient at that point (from flow to static position or its reverse), due to change in velocity head and friction losses.

Fig. 28 shows diagrammatically such typical normal pressure change, for opening and closing, respectively, at a point near the gate of the Drum pipe line. For purposes of comparison the normal line for closing (shown in the upper left part of Fig. 29) has been replotted on the right of one of the gauge charts.

In considering wave pulsations due to gate motion, they must be considered and measured with reference to such a normal curve. For instance, a wave form of Type 5, Fig. 30, superimposed in such a normal curve would give the result shown at the right of Fig. 29. The writer thinks that this feature has sometimes been overlooked by investigators in the past, and the magnitude of the wave pressure has been considered with reference to static conditions rather than with reference to normal, as defined above.

Herein, wherever pressure is noted as above or below "normal" the writer refers to the pressure variation above or below the normal curve, as defined above, and all values of h are so taken.

Discussion of Results.—The last column of Table 1 shows the theoretical values of p for each experiment, where p, in pounds,

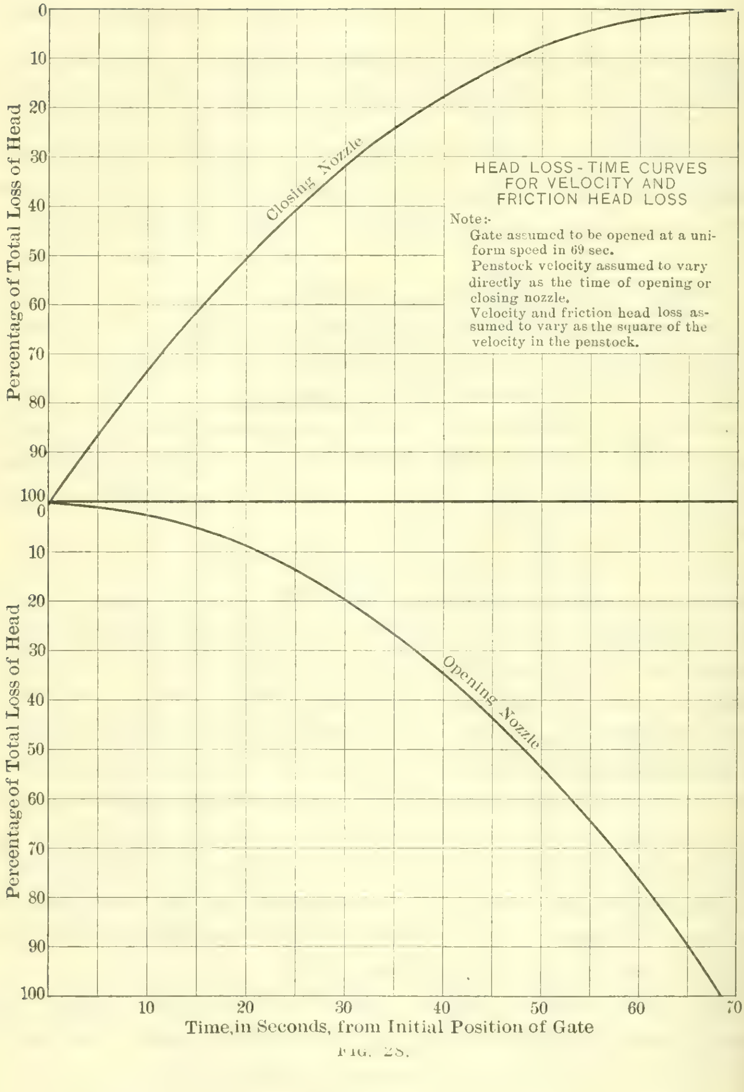

HEAD LOSS - TIME CURVES
FOR VELOCITY AND
FRICTION HEAD LOSS
Note:-
Gate assumed to be opened at a uniform speed in 69 sec.
Penstock velocity assumed to vary directly as the time of opening or closing nozzle.
Velocity and friction head loss assumed to vary as the square of the velocity in the penstock.
Closing Nozzle
Opening Nozzle
Percentage of Total Loss of Head
Percentage of Total Loss of Head
Time, in Seconds, from Initial Position of Gate
0
10
20
30
40
50
60
70
80
90
100
0
10
20
30
40
50
60
70
80
90
100
10
20
30
40
50
60
70

FIG. 28.

= 0.434 h, and where h has been calculated from Equation (2). The values of h have been converted into pounds for ready comparison with the gauge readings. In comparing such calculated values with the average of observed values, the agreement is thought to be remarkably close, both for opening and closing waves. These results certainly seem to substantiate the formula advanced, not only at the gate, but for any other point in the line, and for varying velocities. The time element shown in Table 1 is the time (after the gate begins its motion) when the maximum effect was observed, and this is noted in order that the correct position of the normal at the same instant may be taken.

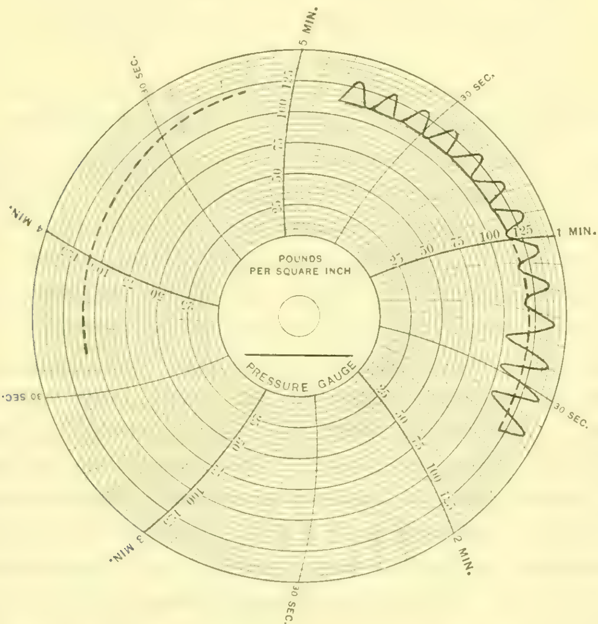


FIG. 29.

Up to the time of this study, in endeavoring to use the formulas heretofore advanced, the writer had always taken the value of V in such formulas as the velocity of flow past the point at which the pressure was desired. That this cannot be the case is very clearly brought out by the results obtained at Gauge No. 1, where, due to the arrangement of nozzles, the velocity of flow was practically constant; at the same time, the wave pressure can be seen to have

varied almost directly with the number of gates opened, or, in other words, with the velocity in the main body of the pipe line.

Wave Synthesis.—In reviewing the charts, it was noted at once that they were quite different in form for points away from the gate from any wave curves previously obtained by other investigators (so far as the writer knows). Previous wave forms, in general, have shown a period of normal pressure between each super-normal and sub-normal crest. In an endeavor to account for this, a study of probable wave forms was made. The writer has always felt that the synthetical method advanced by Professor Joukovsky (which is clearly explained by Miss Simin*) was very useful in studying probable wave effects, and such study in the past has been very helpful to him.

A number of studies by such method were made on the probable wave forms which could be expected in conformity with the conditions existing in the Drum penstock. By such study, in addition to explaining the foregoing form difference, a number of features were discovered which the writer believes to be of considerable interest.

Figs. 30 and 31 cover one of a series of these studies, and have been plotted for Gauge Point No. 2. It is to be noted that the varying wave forms in these diagrams are correct to horizontal scale only, and that the vertical scale is a varying one simply for convenience in plotting. Although the wave forms given, therefore, are characteristic, they are not to scale in magnitude. In Fig. 31 have been built up sixteen succeeding forms of wave which would be obtained with sixteen assumptions of time of gate closure, each varying by ½ sec. From such a series the interesting discovery was made that the series of wave forms is a recurring one every 7 sec. In other words, gate closures of 7, 14, or 21 sec. will have the same corresponding wave form (the magnitude, of course, will decrease with each succeeding length of the time of gate opening). As 7 sec. is equal to $\frac{4\ L}{a}$, it will be seen, as might have been expected, that the wave form is influenced only by the phase in which a partial wave meets on its return to the gate a new wave "just starting."

* *Proceedings*, Am. Water Works Assoc., 1904, p. 341.

WAVE FORM DIAGRAMS FOR VARIATION IN TIME OF GATE CLOSURE, GAUGE POINT NO. 2

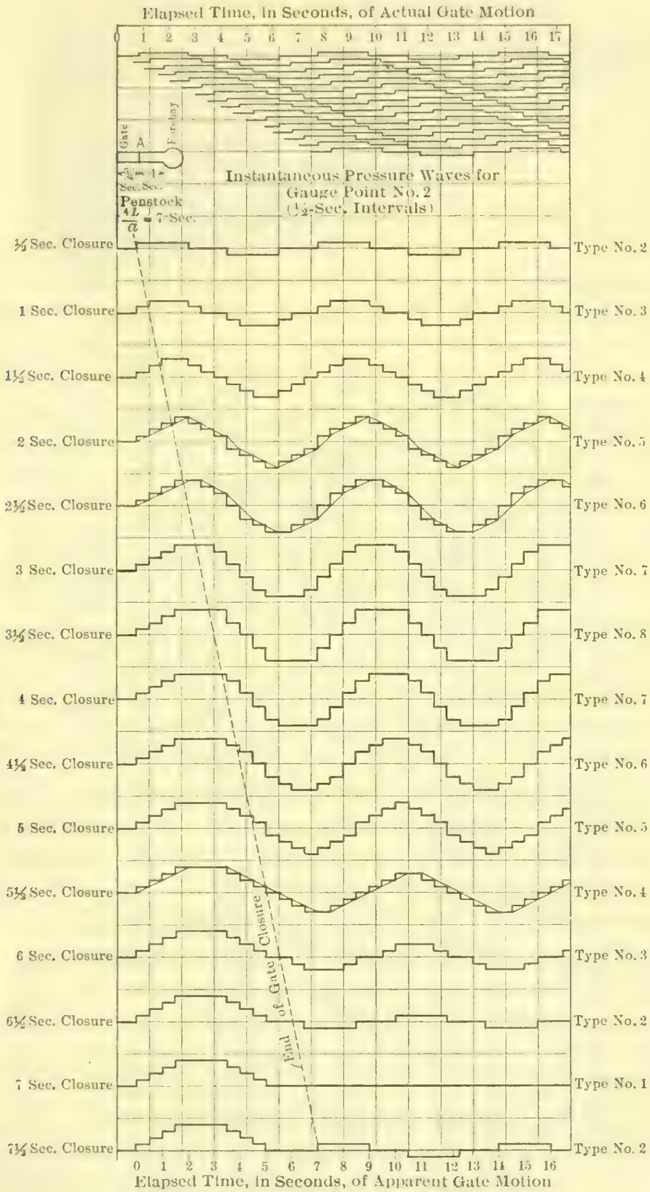


FIG. 30.

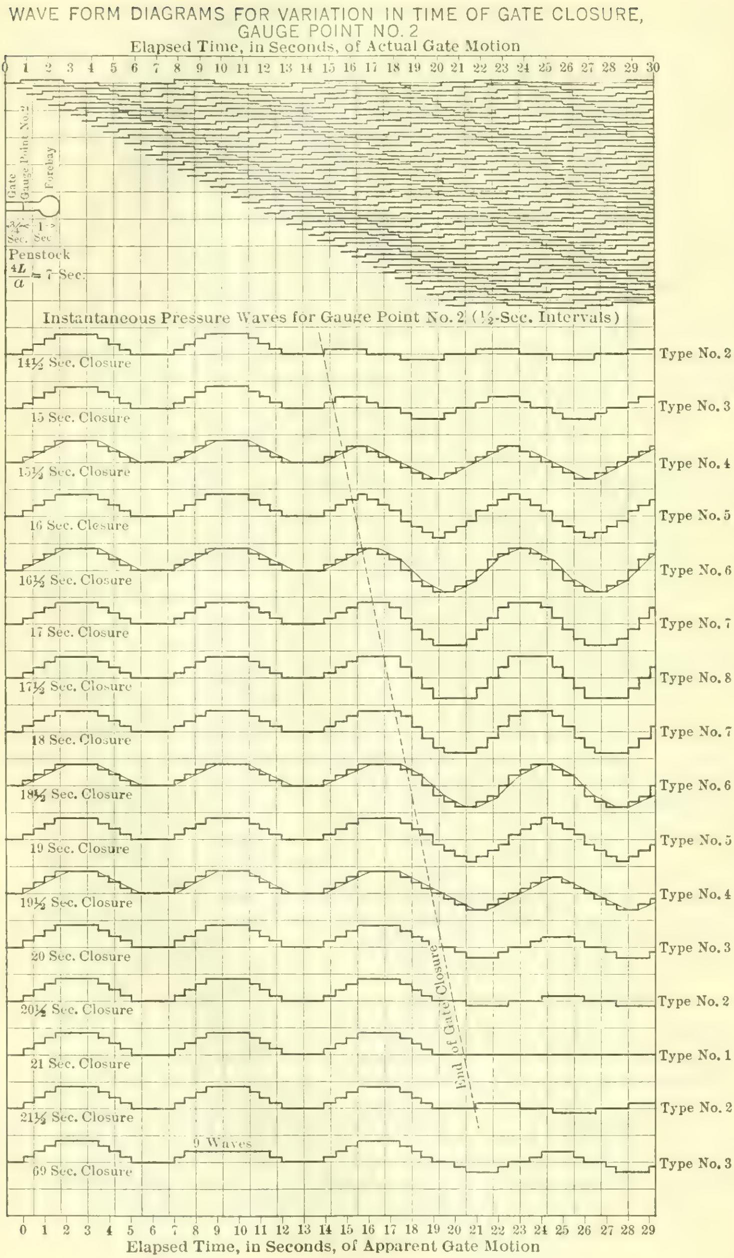

WAVE FORM DIAGRAMS FOR VARIATION IN TIME OF GATE CLOSURE,
GAUGE POINT NO. 2
Elapsed Time, in Seconds, of Actual Gate Motion
0 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21 22 23 24 25 26 27 28 29 30
Gate
Gauge Point No. 2
Forebay
3/4 Sec.
1 Sec.
Penstock
4L/a = 7 Sec.
Instantaneous Pressure Waves for Gauge Point No. 2 (½-Sec. Intervals)
14½ Sec. Closure
Type No. 2
15 Sec. Closure
Type No. 3
15½ Sec. Closure
Type No. 4
16 Sec. Closure
Type No. 5
16½ Sec. Closure
Type No. 6
17 Sec. Closure
Type No. 7
17½ Sec. Closure
Type No. 8
18 Sec. Closure
Type No. 7
18½ Sec. Closure
Type No. 6
19 Sec. Closure
Type No. 5
19½ Sec. Closure
Type No. 4
20 Sec. Closure
Type No. 3
20½ Sec. Closure
Type No. 2
21 Sec. Closure
Type No. 1
21½ Sec. Closure
Type No. 2
9 Waves
69 Sec. Closure
Type No. 3
End of Gate Closure
0 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21 22 23 24 25 26 27 28 29
Elapsed Time, in Seconds, of Apparent Gate Motion

FIG. 31.

From this diagram it is seen that certain wave forms have a period of normal pressure between super-normal and sub-normal crest, and that others do not. It is to be further noted, on Fig. 30, that, for a period of gate closure less than $\frac{2L}{a}$, all the wave forms which occur have such normal periods. Fig. 30 has been drawn to show the forms occurring in the first cycle, that is, for gate opening of periods ranging from 0 to $\frac{4L}{a}$. For the first cycle the wave forms are somewhat different from those for any succeeding cycle.

As previous experimenters have used very short periods of gate closure, the resulting wave forms have always been confined to this portion of the series.

In connection with Figs. 30 and 31, it is to be noted that the portion of the wave shown to the left of the dotted diagonal line across the diagram represents the wave form during the gate motion, and that portion to the right represents the wave form after the gate motion has been completed. It should be noted that no sub-normal pressures can occur previous to the completion of gate motion; but, after a motion is completed, the sub-normal pressure is practically of as great a magnitude, but with reverse sign, as the super-normal pressure. This agrees fully with the charts obtained in the tests.

An inspection of the test charts, however, failed to show any wave crests until the gate had been approximately three-quarters closed, and this is explained by the fact that any greater opening permits reflected waves to pass out through the gate. Such failure to reflect secondary waves at the gate would eliminate wave crest previous to the completion of gate motion, as a little investigation of the diagram, Fig. 31, will show. At about three-quarters gate motion, reflection apparently begins to occur. It is to be borne in mind in this matter that the nozzle has only a diameter of approximately $8\frac{1}{2}$ in., whereas the penstock ranges from 72 to 52 in. Thus, the full gate opening is only a very small portion of the cross-section of the moving water column. Had the experiments been made on the ordinary gate, having an opening equal to the full cross-section of the pipe line, the probabilities are that practically no reflections would have occurred until the very last portion of the gate motion. Such results would give an apparent wave form similar to that on which

Mr. Warren has based his conclusions. Although there is no limit in magnitude (except that for instantaneous gate closure, $\frac{a\,V}{g}$) for the portion of the wave formed during the gate motion, the studies herein submitted will show that certain ratios of length of penstock to time of closure should tend to damp and reduce both the magnitude and the number of the waves after the gate closure has been completed. When it is seen—as by the charts—that the pulsations may last from 5 to 10 min. in the ordinary penstock, it will be at once apparent that the ultimate destructive effect on any line might be importantly reduced by choosing a proper time of gate closure.

Below the sixteen forms studied in Fig. 31, is shown the form for the actual gate period, 69 sec. This corresponds to Wave Type 3. Although this might apparently be considered nonconformative to the actual chart wave, it will be noted, from the curve of gate opening, that the gate does not close quite uniformly, but has an increasing rate of closure at the end which might draw the wave into a different phase from that which would otherwise be expected. A final rate of gate motion corresponding to a time of opening of 68 sec. should apparently give a wave form corresponding to Wave Type 5, Fig. 31, which is more nearly what was actually obtained.

From this it would seem that by a slight adjustment of the time of gate opening between any 65 and 71 sec., some better form of wave might be obtained in the Drum line than that actually now existing.

Amplitude of Vibration.—Finally, a study was made to ascertain, if possible, how closely the formula for velocity of wave propagation

$$a = \frac{12}{\sqrt{\frac{W}{g}\left(\frac{1}{K} + \frac{D}{E\,b}\right)}} \quad \ldots\ldots\ldots\ldots\ldots\ldots (5)$$

could be made to apply to riveted pipe lines. Accordingly, the varying values of a_1, a_2, a_3, etc., were calculated.

In using the formula in connection with the riveted pipe, it was necessary to make allowance for the effect of joints and laps on the rigidity of the pipe, and therefore on the speed of wave propagation, a. Allowance was made for roundabout laps and butt straps by adding the total length of such joints parallel to the axis of the pipe and calculating the time element for an equal length of pipe having

a thickness equal to double the nominal thickness of pipe for lap-riveted work, and for the butt-strap material, using a thickness equal to the nominal thickness plus the thickness of the roundabout strap.

To compensate for the extra rigidity and resistance to diametrical stretch of the longitudinal joint, the additional quantity of metal in tension was computed as uniformly distributed around the pipe, that is, an average thickness, taken over and above nominal, was used.

The results obtained by such method gave a period of 8.76 sec., as compared with a very uniform wave length of 6.95 sec. given by the charts. Although this did not give as close an agreement as the writer had hoped for, the discrepancy is probably due to the fact that nominal instead of actual diameters and thicknesses of pipes were used. This wave period has in no wise entered into the foregoing tables or calculations, but was calculated as a matter of interest. It would be of use in the case of a rapidly moving gate, as entering into the maximum possible value of h.

In conclusion, the writer again calls attention to the fact that the results of these experiments show the importance of allowing for water-hammer in the design of pipe lines, and that, even for slow-moving gates, the matter cannot be safely overlooked. Finally, he hopes that others interested in this subject will discuss these results and pass judgment on them.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

A BRIEF REVIEW OF TRIGONOMETRICAL MATHEMATICAL TABLES, AND A CONTEMPLATION OF THE SPECIFICATIONS FOR TRIGONOMETRICAL TABLES FOR GENERAL USE*

By Virgil A. Eberly, Assoc. M. Am. Soc. C. E.

The writer has felt the need of tables of natural trigonometric functions which cannot now be obtained on the market, and takes this opportunity of explaining this need, which, if generally concurred in, may influence the prospective author to publish tables conforming to the specifications following.

A review of mathematical tables was made by the British Association for the Advancement of Science in 1873, and was very complete. The writer has recently searched the United States Congressional Library and the Library of the Naval Observatory, at Washington, D. C., the latter being exceptionally rich in its collection of mathematical tables. A great many of the tables described in the 1873 report may be seen there, as well as many abridged tables for various purposes, multiplication and conversion tables, and trigonometric tables produced since 1873. A few trigonometric canons produced since 1873 stand out prominently on account of the evident labor expended in

* This paper will not be presented for discussion at any meeting, but written communications on the subject are invited for subsequent publication in *Proceedings* and with the paper in *Transactions*.

compiling them, and on account of the departure from what might be called established practice. The authors of these are:

(*a*). H. Andoyer (1911).
(*b*). J. Bauschinger and J. Peters (1910-11).
(*c*). C. Bremiker (1887).
(*d*). Chamber's Mathematical Tables, by J. Pryde (1899).
(*e*). Mrs. Emma Gifford (1914).
(*f*). J. Peters (1911).
(*g*). G. Rheticus, Opus Palatinum, by W. Jordan (1913).
(*h*). C. E. Raymond (1915).
(*i*). R. Shortrede (1903).

As compared with the older and fundamental tables, these, of course, have much better typographical arrangement and clearness, but it is interesting to note that some of them are lacking, as differences for tabular results are not listed as extensively as in the celebrated "Magnus Canon" (1596-1607), of G. Joachim Rheticus, the greatest of all the table computers, to whom is also due the canon of sines by Pitiscus (1613). The latter are rare, as indicated by a price of £36 15s. and £21, respectively.

Other tables with which American engineers may be more familiar are: Vega's logarithmic tables; Bruhn's logarithmic tables; Hutton's tables of logarithms of Numbers 1 to 108 000, and tables of logarithmic and natural trigonometric functions, etc.; Searles and Nagle's handbooks; etc.

Hutton's and Pryde's productions are very valuable, as they contain several tables which are not found in the books which are more prevalent at the present day. In the 1822 edition of Hutton may be found a very valuable bibliography of tables, if the reader cares to search for information relative to this subject. Other sources of information are: Glaischer's article on "Mathematical Tables" in the 11th Edition of the Encyclopædia Britannica; and William Wesley and Son's* Natural History and Scientific Book Circular No. 147.

The writer submits the following propositions which should govern in the future preparation of trigonometric canons:

Proposition I.—Ninety degrees to the quadrant is and has been used so generally that it is necessary to adhere to this use.

* Booksellers and Publishers, 28 Essex Street, Strand, London, W. C.

Proposition II.—The degree should be decimally divided. Decimal division of the degree is the best for rapid computation, especially where the computations are extended, as in triangulation work. Arguments for such use may be seen in the preface of Professor C. E. Raymond's "Field Manual." In addition to these arguments, the following may be stated: it is easier (*a*) to secure values where interpolation is necessary; (*b*) to average a set of readings of angles; (*c*) to write an expression for an angle; (*d*) to avoid confusion with the abbreviations for feet and inches; (*e*) to plot curves on decimally divided curve sheets; etc.

Proposition III.—Differences should be listed in a column adjacent to all values given, preferably set half way between horizontal lines, in smaller and somewhat lighter type.

Proposition IV.—The tables should be seven-place. Computations should be carried out one place farther than field measurements, and, consequently, seven places is not too great a refinement for steelwork, triangulation, etc. Besides, if one uses a computing machine, seven-place functions are as easily manipulated as five-place.

Proposition V.—The proposed book would be one of natural functions only. Owing to the fact that no engineer who wishes to accomplish computations speedily will attempt to work without a computing machine, logarithmic functions should not be tabulated, as they are more cumbersome than natural functions when using a computing machine. In addition, natural functions have the advantage that actual figures for each partial computation are had, and an observation check as to the relative size of the part of the triangle being solved is readily made.

Proposition VI.—The book should be thumb-indexed. Page numbers should be omitted, as they are apt to be confused with degree numbers. Each page should contain 1°, with arguments to 0.01, except as noted under the following proposition.

Proposition VII.—The book should have four canons, as follows:

Canon I.—(Semi-quadrantal arrangement, 0° to 44° at top of page, 45° to 89° at bottom of page.)

Sine—Difference—Coversine (difference serving for both);

Chord—Difference;

Co-chord—Difference;

Versine—Difference—Cosine (difference serving for both).

Canon II.—(Quadrantal arrangement, 0° to 89° at top of page for tangent and secant, 0° to 89° at bottom of page for cotangent and cosecant.)

Tangent (cotangent same tabular value)—Difference;

Secant (cosecant same tabular value)—Difference.

Canon III.—(Quadrantal arrangement, 0° to 89° at top of page.)

Chord of the supplement—Difference.

Canon IV.—(Biquadrantal arrangement, 0° to 179° at top of page.)

Arc in terms of radius—Difference;

Arc in terms of circumference—Difference;

Area of segment in terms of circumscribing square—Difference.

Canon II should have arguments to 0.001° between 84° and 90°. Where the versine approaches zero, the cosine approaches unity, and the secant (also cosecant, as it is the same tabular value) approaches unity, values should be listed as indicated by the notation used in Merriman's "Civil Engineers' Pocketbook," page 1295, thus: 0.0_31259 indicates that the 0_3 is to be replaced by three ciphers. As usually tabulated, these values are not comparable with other values listed in a seven-place table, because of the succession of similar figures.

Proposition VIII.—As indicated previously, there would be required twelve columns in Canon I (including the arguments at each side of the page), which would not be exceeded by any of the other canons, and, therefore, would govern in the size of page required. For an office book, therefore, a relatively large book would be necessary, in order to tabulate twelve columns for one hundred arguments. If printed on heavy paper, in large type, such a book would have the advantage, however, of remaining open when laid flat on a desk or table.

In such a book of tables as outlined herein, there would be approximately 294 000 printed values, of which 65 334 would be arguments, and 229 656 would be tabular values which would have to be computed.

With about 1 200 printed values per page, the book would require about 246 pages. If, however, each page showed a separate degree

(except for Canon II, 84° to 90°), a book of about 459 pages would be required. The book should be provided with an explanation of the use of the tables with the use of the various computing machines; and especially of the values tabulated under "Area of segment in terms of the circumscribing square."

A page or two, as necessary, might be devoted to a conversion table from minutes and seconds to decimals of a degree.

The writer submits this paper with the belief that such a book of tables is not available, even in the usual form of degrees and sixtieths (minutes), and that engineers will gradually see the merits of the decimal division of the degree, just as they have seen the merits of the decimal division of the foot.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

THE HELL GATE ARCH BRIDGE AND APPROACHES OF THE NEW YORK CONNECTING RAILROAD OVER THE EAST RIVER IN NEW YORK CITY

By O. H. Ammann, M. Am. Soc. C. E.

To be Presented November 21st, 1917.

Synopsis.

The principal object of this paper is to present an account of the design and construction of the Hell Gate Arch Bridge over the East River in New York, a structure of imposing magnitude, with unusual features and details of unprecedented size which mark a decided advance in bridge engineering.

The Hell Gate Bridge, which forms part of the New York Connecting Railroad, is the greatest arch bridge built to date, having a span of 995 ft. $1\frac{3}{8}$ in. between centers of bearings and 1 017 ft. between faces of abutments, and a total height of 305 ft. above mean high water. It carries four railroad tracks on a heavy ballasted floor. Its principal features, besides its great span and capacity, are the exceptional size and weight of its individual members and riveted connections, the use of special high-carbon steel, the unusual method of erection, and the monumental towers forming the abutments, one of which rests on a deep and difficult pneumatic caisson foundation.

Note.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings* and, when finally closed, the papers, with discussion in full, will be published in *Transactions*.

The introductory part of this paper is devoted to the object, history, and a brief general description of the New York Connecting Railroad.

The Hell Gate Bridge is the result of many years of careful and laborious studies involving the design of different types of bridges. These various preliminary designs are briefly described and critically discussed, and the conditions which led to the final adoption of the spandrel-braced arch type are set forth.

The description of the Hell Gate Bridge, as built, is confined to the more important features and details, with particular reference to the reasons and considerations which governed their design, and with a further view to bring out their merits and illustrate the progress made in bridge design in recent years. Typical details are shown in the accompanying illustrations and therefore need no particular description.

With due consideration of the fact that the strength and durability of a bridge are gauged by the strength and efficiency of its details, most careful attention has been given to their design, and a number of important improvements over common practice can be recorded. Among the details deserving special mention are the compact closed section of the main arch rib, the extraordinary rigid bracing, the efficient latticing of compression members, the full splicing of compression joints, and the provision of heavy braking girders to relieve the floor-beams from stress due to the braking and traction forces.

It has been the aim of the administration of the Railroad Company to produce a first-class railroad in every respect. This principle has been observed throughout the design and construction of all its structures. The design has been governed by rules and specifications especially prepared by the consulting engineer. These differ in many respects from common specifications, and contain many original features. Extracts from these rules and specifications are given in this paper, with critical remarks on the selection of the material, assumptions of the various loads and forces, and of the permissible unit stresses.

Among the original features of these specifications may be mentioned a new formula for impact which, in combination with apparently high permissible unit stresses, is applicable to the design of bridges of any length of span or any capacity, and secures in each case well-proportioned structures.

The paper further contains a detailed account of the essential features of the fabrication and erection of the Hell Gate Bridge which called for the highest grade of workmanship, required special tools and machinery of exceptional size, and involved unusual erection problems. This account is intended to illustrate the advance in the fabrication and erection of large bridges of the riveted type. Especial mention may be made of the complete assembling of the arch trusses at the shop, and the extensive utilization of parts of the permanent structure for erection purposes.

The approaches to the Hell Gate Bridge, which comprise the East River Bridge Division of the New York Connecting Railroad, consist of several heavy truss bridges, a double-leaf bascule bridge, about 10 800 lin. ft. of plate-girder viaducts with concrete piers, and about 3 200 lin. ft. of embankment between high retaining walls and concrete arches. Space does not permit of their detailed description, but their unusual and original features, and the reasons and conditions which governed their design, are related.

The calculation of stresses in the arch trusses, as given in Appendix A, although presenting no new theory, will be found of value to those who may have to solve similar problems. It illustrates the application of the modern theory of elasticity in the shortest and most convenient form.

The writer is under obligation, for permission to present this paper and for valuable information, to Gustav Lindenthal, M. Am. Soc. C. E., the Consulting and Chief Engineer, to whose plans and direction the successful completion of the Hell Gate Bridge and Approaches is due.

For convenience of reference and discussion, the paper is written under the following principal headings:

1.—Object and History of the New York Connecting Railroad;
2.—General Description of East River Bridge Division;
3.—Development of Design of Hell Gate Bridge;
4.—General Arrangement, Proportions and Cost of Arch Bridge;
5.—Masonry Towers and Foundations;
6.—Details of Design, and Weight of Steel Superstructure of Hell Gate Bridge;
7.—Camber and Deformation of Arch Trusses;
8.—Material;
9.—Loads and Unit Stresses;

1.—Object and History of the New York Connecting Railroad.

Object.—The New York Connecting Railroad is a part of the comprehensive plan of the Pennsylvania Railroad, inaugurated 15 years ago by the late A. J. Cassatt, then President of that Company, and which had for its principal objects the extension of the line from New Jersey into New York City and direct rail connection, for passengers and freight, with Long Island and the New England States.

The parts of this general project which established entrance into New York City, and connection with the Long Island Railroad, comprising the tunnels under the North River, Manhattan Island, and the East River, the great passenger terminal in Manhattan, a large terminal yard, called "Sunnyside Yard", in Long Island City, a freight terminal yard, at Greenville, N. J., and various other improvements have been fully outlined and described in detail.* (See also the map, Fig. 1).

The New York Connecting Railroad establishes a physical connection between the Pennsylvania Railroad and the New York, New Haven, and Hartford Railroad Systems, and thus provides continuous rail communication, through New York City, between Canada, the New England States, and the South and West.

The connection with the New Haven Line is made at Port Morris in The Bronx. From there, the Connecting Railroad crosses the East River, *via* Wards and Randalls Islands, and joins the Pennsylvania Railroad at Sunnyside Yard in Long Island City, whence the trains pass through the East River Tunnels to the Pennsylvania Station at 34th Street, in Manhattan. This connection *via* Sunnyside Yard,

* *Transactions*, Am. Soc. C. E., Volumes LXVIII and LXIX (1910).

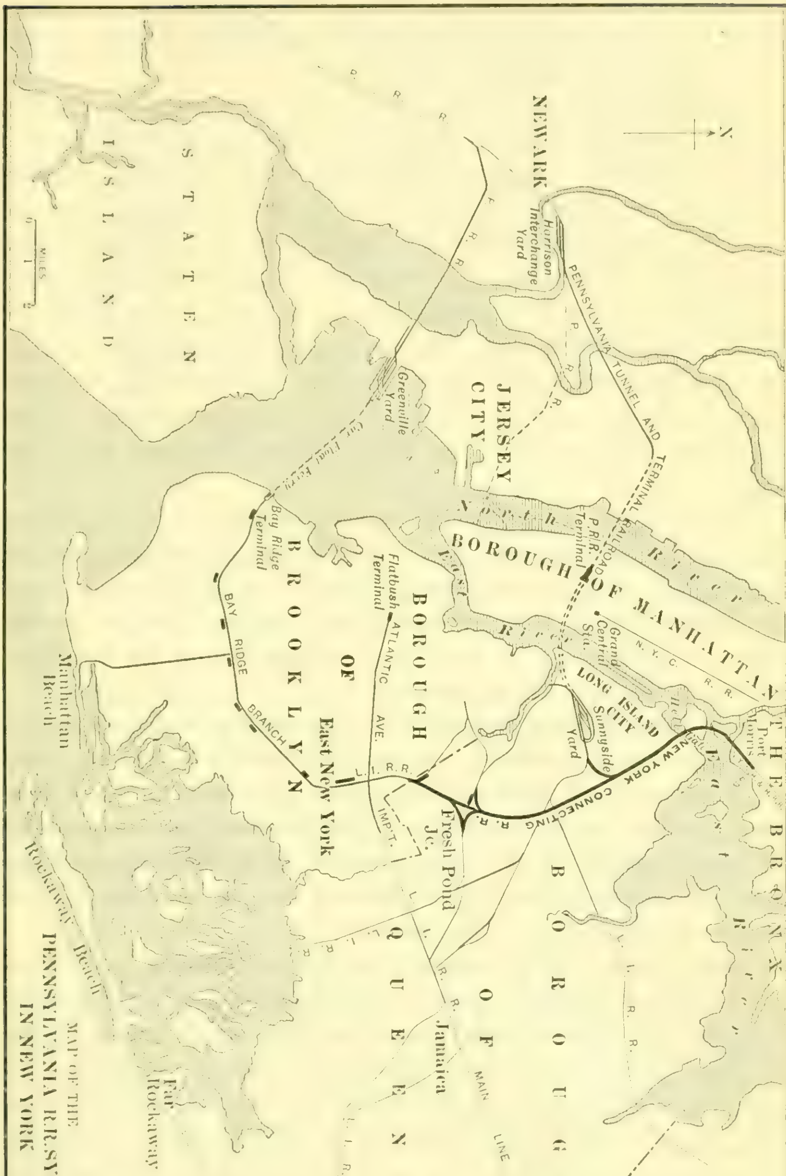

MAP OF THE
PENNSYLVANIA R.R. SYSTEM
IN NEW YORK
NEWARK
Harrison Interchange Yard
PENNSYLVANIA TUNNEL AND TERMINAL RAILROAD
JERSEY CITY
Greenville Yard
Car Float Ferry
STATEN ISLAND
MILES
North River
BOROUGH OF MANHATTAN
P. R. R. Terminal
Grand Central Sta.
N. Y. C. R. R.
East River
Harlem River
THE BRONX
Port Morris
LONG ISLAND CITY
Sunnyside Yard
NEW YORK CONNECTING R. R.
Fresh Pond Jc.
BOROUGH OF BROOKLYN
Bay Ridge Terminal
BAY RIDGE BRANCH
Flatbush Terminal
ATLANTIC AVE. IMP'T.
L. I. R. R.
East New York
BOROUGH OF QUEENS
Jamaica
MAIN LINE
Manhattan Beach
Rockaway Beach
Far Rockaway

Fig. 1.

however, is intended for passenger trains only, inasmuch as the tunnels under the North and East Rivers and the Manhattan Station can now accommodate only passenger traffic.

The freight connection will be made by another branch of the New York Connecting Railroad, which after crossing the East River at Hell Gate, passes through Long Island City, Woodside, Winfield Junction, and Middle Village, to Fresh Pond Junction, near the Brooklyn Borough line, where it connects with the Bay Ridge Branch of the Long Island Railroad. From Bay Ridge, the freight cars are transported on car-floats across New York Bay to the Greenville Yards of the Pennsylvania Railroad in New Jersey. Eventually, a tunnel may be driven under New York Bay from Bay Ridge to Greenville, which would provide also for freight by continuous-rail connection from New England to the South and West through New York City.

Besides this through freight traffic, the New York Connecting Railroad will accommodate the bulk of the freight transportation to and from the large manufacturing districts which have been developed during recent years in Brooklyn and Queens. For this purpose a number of freight yards have been established along the Bay Ridge Branch of the Long Island Railroad.

Although not doing away entirely with car-float and ferry service, the New York Connecting Railroad will greatly relieve the inner waters of New York Harbor of this traffic, particularly of the obstructive car-floats which now transfer freight from the Pennsylvania Railroad terminals on the Jersey shore of the Hudson River, up the East River to the New Haven Railroad freight terminals at Port Morris and Oak Point, in The Bronx.

Traffic.—The early plans contemplated a double-track line. Later, it was realized that the probable future development of an extensive passenger and freight traffic between the New Haven and the Pennsylvania Systems would soon require four tracks. Estimates also proved that it would be much more costly to add two tracks in the future than to build a four-track line in the first place. The entire portion from Port Morris to the point in Long Island City where the two passenger tracks branch off toward Sunnyside Yard was built therefore for four tracks, the remainder being for two tracks only.

The bulk of the traffic over the New York Connecting Railroad will be freight. Passenger traffic will be limited, for the near future at least, to about 80 trains daily, 40 in and out of the Pennsylvania Station, on account of the restricted capacity of that station. Most of the passenger trains of the New Haven Railroad will continue to run into the Grand Central Station of the New York Central Railroad.

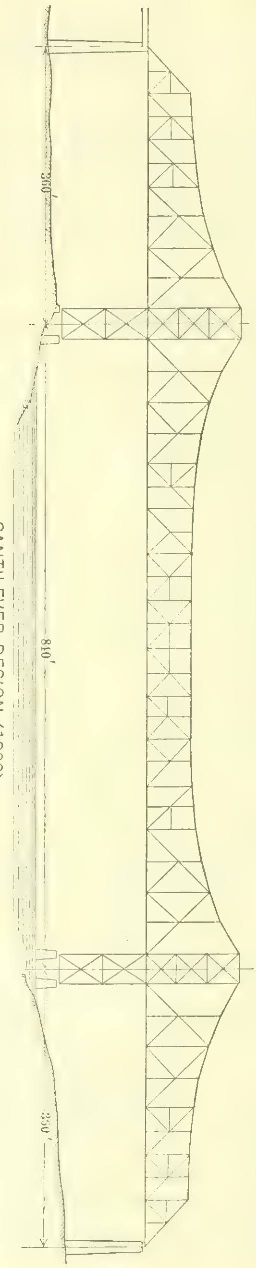
No local passenger stations are provided on the New York Connecting Railroad between Port Morris and the Pennsylvania Station in Manhattan, as no local passenger traffic is to be carried between points within the city limits.

The New York Connecting Railroad will be operated electrically by the single-phase alternating-current system now used on the New Haven Railroad. The current will be delivered by an overhead catenary system supported by the steel superstructure. As the Pennsylvania Railroad, between Sunnyside Yard and Manhattan Transfer, *via* the Pennsylvania Station, has the direct-current system with third-rail, the passenger trains will have to change locomotives at Sunnyside Yard or be provided with locomotives equipped with contact shoes and control in order to enable them to run over both systems. On all bridges and viaducts of the New York Connecting Railroad there are footwalks for the use of employees.

Realizing the opportunity for a cheap highway connection between the Boroughs of The Bronx and Queens, Mr. Lindenthal conceived and investigated the plan of carrying a 60-ft. highway, with trolley tracks, on an upper deck of the bridges and viaducts. Should the necessity arise, it would be entirely feasible to carry out such a plan at a comparatively moderate expenditure. It would provide the city with a magnificent boulevard connecting two densely populated districts.

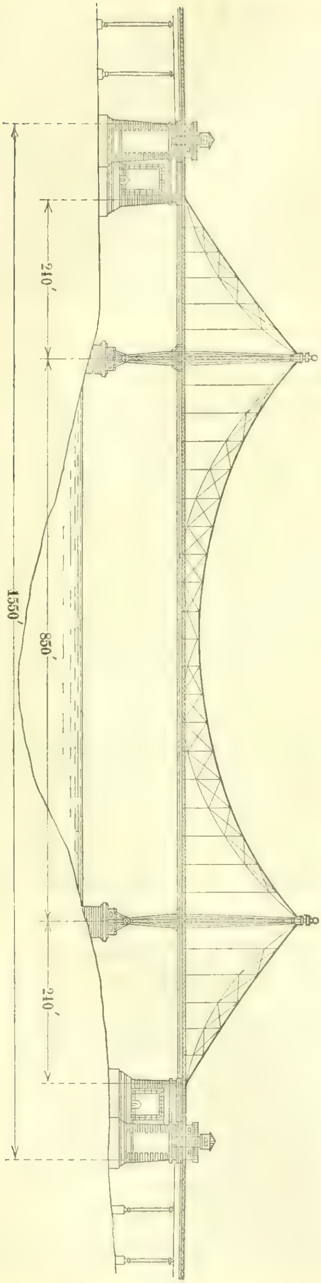
History.—The history of the New York Connecting Railroad is linked up with various plans and enterprises which had as their object improved transportation facilities in and around the City of New York. The project of crossing the East River at Hell Gate took concrete form for the first time in 1892, when a charter was secured by the late Oliver W. Barnes, M. Am. Soc. C. E., a well-known civil engineer, to build a double-track railroad connecting the New York Central Railroad with the Long Island Railroad.

Plans for this line with a cantilever bridge of 840 ft. span (Fig. 2) over the East River at Hell Gate were worked out by the late Alfred



CANTILEVER DESIGN (1900)

FIG. 2.



SUSPENSION BRIDGE DESIGN (1904)

FIG. 3.

P. Boller, M. Am. Soc. C. E., as Consulting Engineer, and construction was to be started in 1900. At this time the Pennsylvania Railroad had mapped out its scheme for extending its line from New Jersey into Manhattan. The North River was to be crossed on a suspension bridge of 3 100 ft. span, which had been designed and promoted previously by Mr. Lindenthal. The bridge would have accommodated ten railroad tracks, rapid transit and highway traffic, and would have required the participation of all the other railroads terminating in New Jersey. The bridge was to be connected by an approach descending with a 2% grade to a tunnel under 42d Street, commencing near Tenth Avenue and extending across Manhattan and under the East River to a connection with the Long Island Railroad System. This tunnel was called the Steinway Tunnel, after the President of the Tunnel Company of which Mr. Barnes was the Chief Engineer. Mr. Lindenthal mapped out to Mr. Barnes a connection of the Long Island System with the New Haven Railroad System in The Bronx on substantially the present line of the New York Connecting Railroad, thus establishing a continuous line through New York City territory.

As the other railroads in New Jersey declined to co-operate in the North River bridge project, this was finally abandoned by the Pennsylvania Railroad in favor of two single-track tunnels, the previous objections to tunnels for train service having by that time been overcome by the introduction of electric operation.

In 1900, the Pennsylvania Railroad acquired control of the Long Island Railroad, and a connection with this road across the East River, and with the New England roads, *via* the New York Connecting Railroad on the line originally mapped out, became, therefore, necessary adjuncts to the plan of entering Manhattan. As a result, the Pennsylvania Railroad, in conjunction with the New Haven Railroad, acquired the charter of the New York Connecting Railroad. Samuel Rea, M. Am. Soc. C. E., then Vice-President of the Pennsylvania Railroad, took charge of the entire project as President of the New York Connecting Railroad. The entire work was under his direction. A. J. County, Assoc. Am. Soc. C. E., was Assistant to the President, and Mr. Lindenthal was appointed Consulting Engineer and Bridge Architect to work out new plans for the East River Bridge and Approaches. The surveys and location in detail, as well as the first

borings for the foundations, were made by Joseph N. Crawford, Consulting Engineer of the Pennsylvania Railroad. Later, this work was turned over to Mr. Lindenthal, who appointed his own staff.

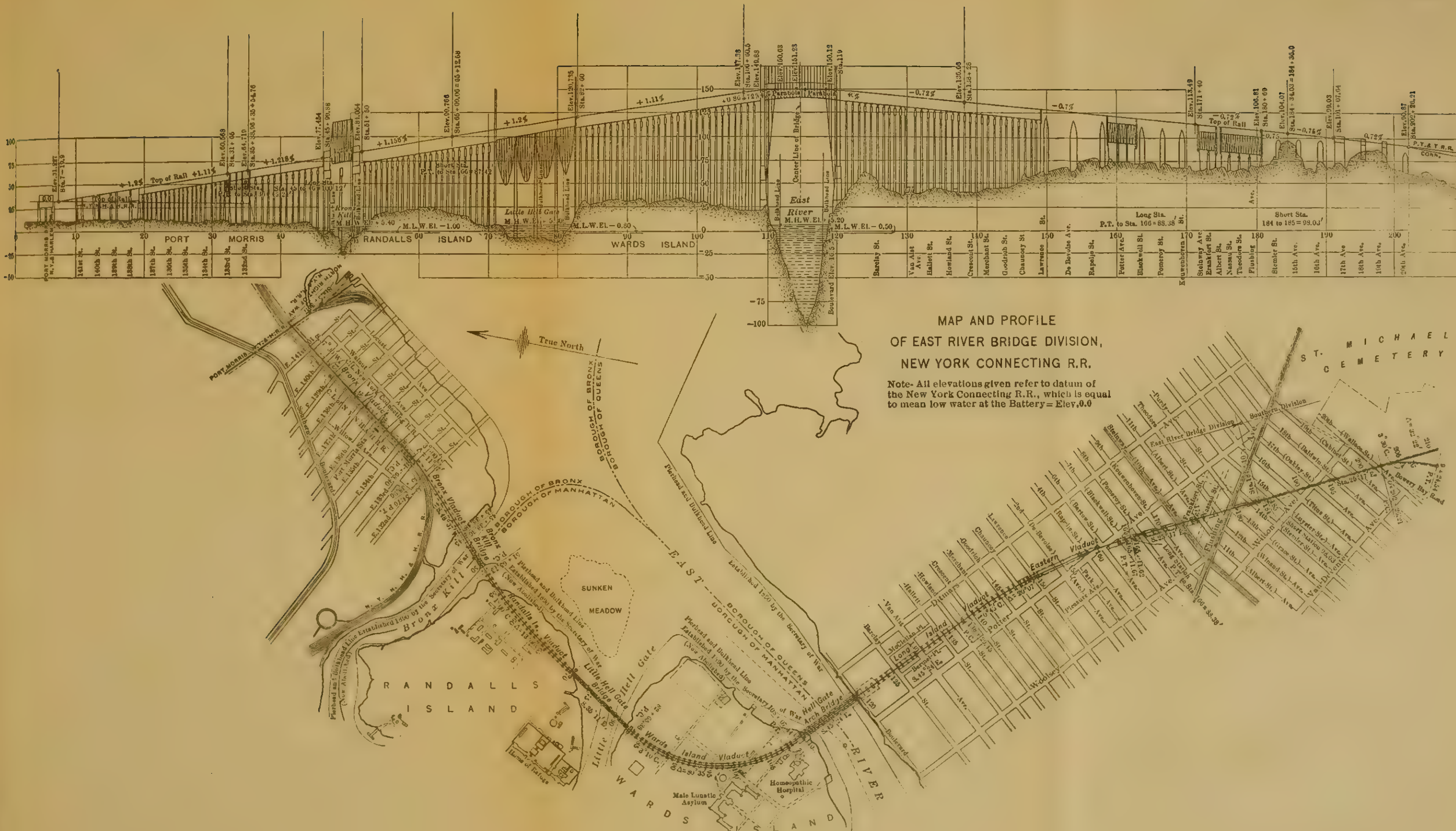
A franchise for the construction of the New York Connecting Railroad was granted by the Board of Rapid Transit Railroad Commissioners of New York (succeeded by the present Public Service Commission for the First District), in February, 1907. On account of the financial stringency in this year, the commencement of actual construction was again delayed until July, 1912, when ground was broken. It was estimated that construction would be completed by January 1st, 1916, but an injunction and unforeseen difficulties in building the foundations for the Hell Gate Bridge delayed the completion for more than a year. Traffic was established in March, 1917.

2.—General Description of East River Bridge Division.

Length.—For construction purposes, the New York Connecting Railroad, 8.96 miles long, has been separated into two divisions. The "East River Bridge Division", which extends from the junction with the New Haven Railroad in The Bronx across the East River to Stemler Street in Long Island City, a distance of 3.38 miles, was designed by Mr. Lindenthal, as Consulting Engineer and Bridge Architect, and executed by him as Chief Engineer. The "Southern Division", which comprises the remainder, or 5.58 miles, was in charge of Mr. A. C. Shand, Chief Engineer, and H. C. Booz, M. Am. Soc. C. E., Assistant Chief Engineer. This paper is devoted exclusively to the East River Bridge Division, which consists principally of bridges and viaducts, and has consumed about two-thirds of the entire cost. The plan and profile of this Division are shown on Plate XVI.

Location.—From the junction of the New York Connecting Railroad with the New Haven Railroad, immediately north of the crossing over the Port Morris Branch of the New York and Harlem Railroad (New York Central Railroad), the two roads run parallel in a southwesterly direction on eight tracks to East 133d Street, the four New York Connecting Railroad tracks rising gradually above the New Haven tracks. South of 133d Street, the two westerly or passenger tracks of the Connecting Road cross over the two easterly or freight tracks of the New Haven Road. At this point the New Haven tracks

PLATE XVI.
PAPERS, AM. SOC. C. E.
OCTOBER, 1917.
AMMANN ON
HELL GATE BRIDGE.



MAP AND PROFILE
OF EAST RIVER BRIDGE DIVISION,
NEW YORK CONNECTING R.R.

Note- All elevations given refer to datum of the New York Connecting R.R., which is equal to mean low water at the Battery = Elev. 0.0

turn on a sharp curve toward the northwest, ending in the Harlem River Station and Yard, opposite 125th Street, Manhattan.

The New York Connecting Road continues in the southeasterly direction, crosses Bronx Kill on a double-leaf bascule bridge, and then skirts the east shore of Randalls Island on a viaduct 1 965 ft. long. Another arm of the East River, called "Little Hell Gate", between Randalls and Wards Islands, is bridged over by four skew deck truss spans, of an aggregate length of 1 154 ft.

On Wards Island the line turns to a southeasterly direction with a sweeping curve on a viaduct from 95 to 130 ft. high and 2 654 ft. long. The main channel of the East River is crossed at "Hell Gate" by an arch bridge having a single span of 1 017 ft. between abutment towers, a total height of 305 ft. and a clear head-room of 135 ft. above mean high water.

On the Long Island side of the East River, the road continues in a southeasterly direction, descending partly on a viaduct, and partly on an embankment, from a height of 110 ft. to 30 ft. above ground at Stemler Street, Long Island City, where the Southern Division begins.

The total length of the main line is 8.96 miles, of which 3.73 miles have four tracks and the remainder two tracks. A study of the map and of the river conditions as outlined herein will show at once that this route, crossing the East River at its narrowest point, was the natural one to follow. Any other route would have been much more expensive. A tunnel under the East River at this point would have been a very expensive and hazardous undertaking, and would have deprived the passengers of the picturesque and more comfortable ride over the elevated structure.

Curves and Grades.—The maximum curvature is 4° for a short distance at 133d Street, The Bronx. On Wards Island, the curvature is 3° 10′ for a length of 2 545 ft. All other curves are less than 1 degree. North of Hell Gate Bridge, or against south-bound traffic, the grade is nearly uniform, 1.2% (maximum 1.218%), compensated on curves. This grade was governed by the elevation of the tracks over the East River, where the clear height of 135 ft. above mean high water was prescribed by the War Department; this is the same clearance as that for the other East River bridges. South of Hell Gate Bridge, the tracks descend on a uniform grade of 0.72%, the ruling grade for north-bound traffic.

Quantities and Cost.—The New York Connecting Railroad will be one of the most expensive railroad lines ever built. Its total cost, inclusive of right of way, will be about $27 000 000. Of this, the East River Bridge Division consumed approximately $18 500 000, or $5 500 000 per mile of four-track line. This division required about 500 000 cu. yd. of granite and concrete masonry and 90 000 tons of steelwork.

3.—Development of Design of Hell Gate Bridge.

Introductory.—A great work of art evolves from an idea in the mind of its creator. It is brought on paper or into a more contemplative form and then changed and remodeled. Not until the plans have passed through changes and corrections, and have been submitted to an almost endless series of finishing touches, does the great work attain its perfection.

A great bridge in a great city, although primarily utilitarian in its purpose, should nevertheless be a work of art to which Science lends its aid. An elaborate stress sheet, worked out on a purely economic and scientific basis, does not make a great bridge. It is only with a broad sense for beauty and harmony, coupled with wide experience in the scientific and technical field, that a monumental bridge can be created. Fortunately, the Hell Gate Bridge was evolved under such conditions, and therefore may well be said to be one of the finest creations of engineering art of great size which this century has produced.

As mentioned heretofore, under "History of the New York Connecting Railroad", the first design for the Hell Gate Bridge was made in 1900 by the late Alfred P. Boller. It was a cantilever design with a central span of 840 ft., supported on braced steel towers. (Fig. 2.) The bridge was designed for two tracks only, for a light live load, approximately equivalent to Cooper's E-40, and for open tie flooring.

From 1904, when Mr. Lindenthal was appointed by the Pennsylvania Railroad to work out new plans on more modern lines, until 1912, when actual construction was started under his direction, the design of the Hell Gate Bridge received almost continuous and thorough study, involving the working out of complete designs of several types of bridges and a number of modifications of the type finally adopted.

River Conditions.—The East River is an estuary or tidal stream forming the eastern entrance to New York Harbor from the Atlantic Ocean by way of Long Island Sound. That part immediately east of its confluence with the Harlem River, between Wards Island and the Long Island shore, is the so-called "Hell Gate," which name is due, evidently, to the great dangers and trying conditions to which navigation was formerly exposed at this spot. On account of the presence of many protruding rocks and reefs, the sharp bend of the channel just below Hell Gate, and the rapid tidal currents, which even now attain velocities of 7 miles per hour, collisions and disasters in this locality were of frequent occurrence. These conditions have been greatly improved by the removal of the most dangerous reefs, some of which required the blasting away of enormous quantities of rock. The most famous operation was the removal of "Flood Rock", in 1885, when nearly 300 000 cu. yd. of rock were broken up in one blast.

The river at the Hell Gate Bridge is 850 ft. wide between shore lines and 700 ft. between bulkhead lines, as established by the War Department. Both shores fall rapidly to a greatest depth of 105 ft. below mean high water. The mean tide is 5.7 ft. At present, the channel has a minimum depth of 26 ft., but, eventually, it is to be dredged to a depth of 35 or 40 ft. so as to permit the safe passage of deep-draft vessels. The river traffic is quite considerable, consisting to a great extent of car-floats and tows which are difficult to control.

Types to be Considered.—The river conditions, as outlined, and the great height of the tracks above the water, prohibited physically and economically the construction of any permanent or even temporary support in the river channel, and called for a single river span of at least 850 ft., and of a type that could be erected without the use of falsework in the river. The only types which can be taken into consideration under such conditions are the cantilever or its relative, the continuous truss, the stiffened suspension bridge, and the arch (hingeless, two, or three-hinged).

Judging from prevailing tendencies, most engineers undoubtedly would have considered the cantilever type as best suited to the existing conditions, and it is not surprising, therefore, that the first design was of that type.

The suspension type for railroad bridges is considered by many engineers as unsuitable for spans of less than 2 000 ft., and, perhaps,

very few would have looked to the arch as a suitable type in this case, because it is usually associated with steep rocky shores which afford natural solid abutments and cheap anchorages for erection back-stays.

The span length, required clearance, character of soil, and other local conditions at Hell Gate are such that, in a broad sense, there is little if any difference in cost between the several types mentioned. Whatever differences in cost may be found by comparative designs are largely due to the individual judgment of the designer in the selection of the truss system, material, permissible unit stresses, foundations, and architectural features.

A real economy of the suspension type over the others comes in with spans greater than 850 ft.; an appreciable economy of the arch over the cantilever and suspension types would have been realized with more favorable configuration and character of ground, particularly if the required clearance had permitted the same span length for the arch as for the other types. There was the more reason, therefore, for selecting the type for the Hell Gate Bridge on broader than mere economic principles.

Mr. Lindenthal conceived the bridge as a monumental portal for the steamers which enter New York Harbor from Long Island Sound. He also realized that this bridge, forming a conspicuous object which can be seen from both shores of the river and from almost every elevated point of the city, and will be observed daily by thousands of passengers, should be an impressive structure. The arch, flanked by massive masonry towers, was most favorably adapted to that purpose.

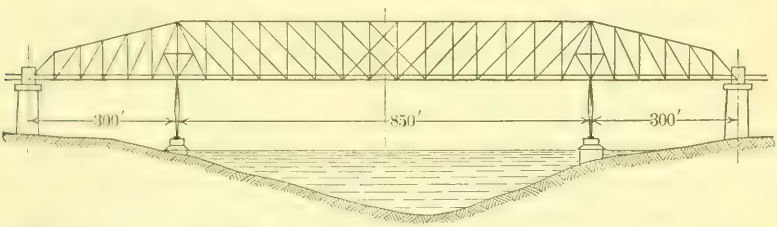
Comparative Designs
of Cantilever, Continuous, and Suspension Types.

In 1904 Mr. Lindenthal made comparative designs of the three types comprising the stiffened suspension type with eye-bar chains (Fig. 3), the three-span continuous truss (Fig. 4), and the three-span cantilever (Fig. 5), all with a central span of 850 ft. and a total length varying from 1 450 to 1 550 ft. The designs were made both for two and four tracks, and open tie flooring. The live load assumed was the Pennsylvania Railroad standard loading of 1904, which is approximately equivalent to Cooper's E-50.

Nickel steel was assumed for the trusses in the four-track designs only, and ordinary carbon steel for the floor system, bracing, and towers

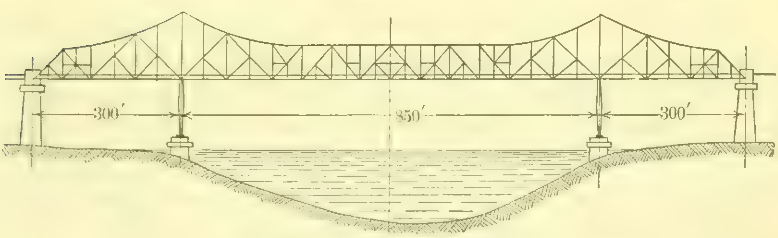
in the four-track designs and throughout for the two-track designs. The estimated weights of steelwork varied from 7 000 to 8 500 tons for double-track and from 11 200 to 14 200 tons for four tracks (if carbon steel had been assumed these weights would have been 14 000 and 17 000 tons, respectively), being least for the suspension bridge and greatest for the cantilever.

The saving in steelwork in the suspension design was partly offset by the greater cost of the anchorage piers, but, under assumed favorable soil conditions, the estimates showed a saving in cost in favor of the latter design. Under the more unfavorable soil conditions actually found on the Wards Island side, the total cost would be more nearly alike for the different designs.



CONTINUOUS TRUSS DESIGN (1904)

FIG. 4.



CANTILEVER DESIGN (1904)

FIG. 5.

Suspension Design.—The system of trusses adopted for the suspension design (Fig. 3) is that of an inverted three-hinged spandrel-braced arch suspended from hinged towers. The upper chord or chain of eye-bars follows very nearly the equilibrium polygon for dead load. The web members and lower chord form, with the main chain, the stiffening trusses.

Owing to the hinge at the center, the system is statically determinate and immune to settlements of the foundations. A similar system,

but without the center hinge, was used by Mr. Lindenthal in his design for the bridge over the St. Lawrence River, at Quebec, made in 1898, and for the Manhattan Bridge over the East River in New York City. This system has been fully described and discussed by him,* and, therefore, will not be further explained here.

The system used in Mr. Lindenthal's design for the Quebec Bridge, made in 1910,† although similar in form, differs from the previously mentioned system in that its two chords or chains form intersecting catenaries equi-distant from the line of equilibrium for dead load. This system is applicable to very long spans only, where the tension in the chains from dead load cannot be reversed by the live load.

In the design for the Hell Gate Bridge a hinge was provided at the center, as it was not known whether solid rock foundations could be obtained at reasonable depth. Wherever the piers rest on unyielding foundations, it is preferable to omit the hinge. In comparison with the bridge types shown in Figs. 4 and 5, the suspension type, with its graceful outlines, possesses unquestionably the advantage of more pleasing and monumental appearance. The anchorage masonry and the main towers give opportunity for architectural treatment. The appearance would be improved by a slight increase in the length of the end span.

In point of rigidity, the cantilever, in general, is superior to the suspension bridge. However, with the system of stiffening trusses selected for this design and the great sag of the chains (one-sixth of the span length), the deflections of the suspension bridge are reduced to about one and one-half times those of the cantilever. This greater deflection, however, is no serious disadvantage in a bridge of such size and capacity, in which the dead load is more than twice as much as the maximum assumed live load and more than four times the live load under average traffic conditions. Moreover, in a suspension bridge, the deflections from live load are free from that jerkiness which is the disagreeable characteristic of deflection in the cantilever bridge system.

Although unusual, the erection of the eye-bar chain suspension bridge, without falsework in the center span, is entirely feasible, and does not present more serious difficulties than that of a cantilever.

* *Transactions*, Am. Soc. C. E., Vol. LV (December, 1905).

† *Engineering News*, November 23d, 1911.

It was particularly for its monumental character, together with its apparently smaller cost, that the stiffened suspension bridge was recommended by the designer for adoption in preference to the other types, and it would undoubtedly have been executed had construction been started at that time and had not certain conditions developed later which led to the study and final adoption of the arch type.

Continuous Truss.—In point of rigidity, the continuous truss (Fig. 4) is superior to the cantilever and suspension type. As regards appearance, however, this design is the least satisfactory. Both for economy and appearance the design could be improved by depressing the top chord so as to make the height at the middle of the center span about two-thirds of that over the main piers, and by using a single instead of the antiquated double web system. In any case, however, the appearance would be that of a utilitarian structure.

The common objection to a bridge of this type is that the trusses are statically indeterminate and may be affected seriously by settlements of the foundations. Where solid foundations can be reached at reasonable depth, this objection, however, is not valid, and this type may be found to be cheaper and more suitable than other types if esthetic considerations are of no importance. The erection of a continuous truss bridge, being similar to that of a cantilever, does not present unusual difficulties.

Cantilever Design.—The cantilever design (Fig. 5) is similar to that shown in Fig. 2, except that slender hinged towers take the place of the unsightly braced towers of the latter design. Although superior in appearance to many existing cantilever bridges, both designs produce the effect of utilitarian structures inherent to cantilever bridges. There is no opportunity for monumental towers or abutments at the ends, because the absence of a large horizontal thrust or pull does not justify a large mass of masonry at those points, as in the case of an arch or a suspension bridge.

The hinged towers in the design (Fig. 5) are a distinct improvement over the braced towers, in that they eliminate dangerous secondary stresses which are caused in the case of the latter. A further decided improvement as regards appearance, rigidity, and permanence, and, therefore, fully justifying the slightly greater expense, would be the substitution of solid masonry piers for the steel towers below the bottom chord of the trusses. Such piers have the further advantage

that the longitudinal forces from braking and traction can be transmitted through them to the foundations on the shortest way and without increasing appreciably the size of the piers. In the design (Fig. 5) the anchor arms have the proper length, whereas in the design (Fig. 2), they are too long, both as regards economy and as the ends are subject to reversal of reaction from live load which causes objectionable "hammering."

Conditions Which Led to Investigation of Arch Type.

In 1905 the line of the railroad on Wards Island was moved farther north, in order to keep it a greater distance from the State Hospital buildings there. This resulted in a sharp 3° 10′ curve extending at both ends almost to the shore line of the island. A long shore span such as would have been required for a cantilever or suspension bridge would have necessitated a still sharper curve, which was not desirable on account of the heavy grade.

Moreover, in the case of a cantilever or suspension bridge, it would have been necessary, in order to keep the span length down to 850 ft., to place the main piers close to the shore lines. On the Long Island side this would have necessitated, at considerable additional expense, the shifting landward of the boulevard which runs along the river shore, and would undoubtedly have been objected to by the City authorities.

These conditions induced Mr. Lindenthal to investigate a single-span arch bridge design. For an arch, the location of the Long Island abutment on the land side of the boulevard was the proper one, the span length of about 1 000 ft. being determined by the clear height of 135 ft. which was required by the Government for navigation and had to be maintained for the full width between the established bulkhead lines. By that time, also, preliminary borings had been completed, giving more accurate information as to the soil conditions. These borings indicated that solid foundations, which are necessary to resist the tremendous thrust of an arch of such great span, could be had at reasonable depth.

Comparison of Arch with Cantilever and Suspension Types.

Comparison in cost with the suspension and cantilever designs indicated a saving in favor of the arch. The estimated weight of steelwork of the arch, using carbon steel only, was about 13 000 tons, as

compared with 14 000 and 17 000 tons for the suspension and cantilever designs, respectively. As actually built, the arch is probably no cheaper than a cantilever or suspension bridge under the same conditions, because the saving in steelwork was practically offset by the greater cost of the more elaborate towers adopted in the final design, and by the greater cost of foundations, due to the unfavorable soil conditions encountered on the Wards Island side.

Had the arch abutments been designed to satisfy only the static requirements, a substantial saving in favor of the arch would have resulted, and, of course, still more so, if it had been possible to give the arch a span of only 850 ft. These conditions, together with the considerations for appearance, finally led to the adoption of the arch type.

The two masonry towers placed at each end of the bridge are an architectural necessity. Without them, the arch would lose much of its monumental character and be reduced to a utilitarian structure similar to the cantilever. The towers, however, have their static function also. With their great weight, they steepen the resultant arch thrust and thereby limit the size of the deep foundations to a minimum. They also facilitated and cheapened the erection of the arch to a considerable extent. Aside from the considerations which favored it in this case, the arch type, as finally adopted, possesses over the suspension and cantilever types the advantages of greater rigidity, its vertical deflection under live load being only about two-thirds of that of the cantilever. The deflections, which are maximum at the quarter-points of the arch span, are not greater than those at the center of a simple span, which is well known as the stiffest type of bridge.

The secondary stresses, also, are small in an arch of the adopted type, and very much smaller than in most cantilever bridges, although this depends very much on the truss system and the method of erection.

The economy of an arch depends largely on the method of erection. An arch is erected either on falsework (which was out of question in this case) or by the cantilever method, being, in the latter case, held by the temporary back-stays. These back-stays may require considerable extra material, and may thus impair the economy of the arch, unless the configuration of the ground is such that they can be made of short ties and anchored cheaply in solid rock.

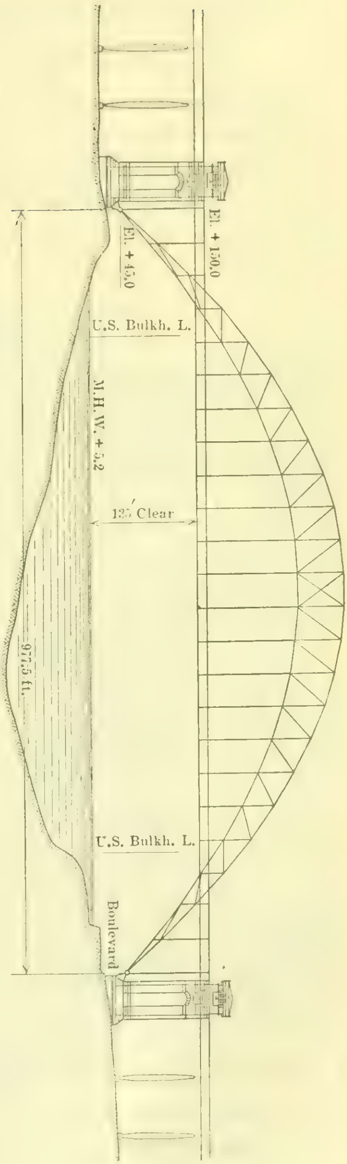
This was not possible in the case of the Hell Gate Bridge. However, the adjoining viaduct spans, the floor system, suspenders, and

other parts of the arch bridge proper afforded in this case ample material to make up the temporary back-stays and anchorages, so that only little extra material had to be used. This cheapened the erection very considerably. The erection on the cantilever principle presents no more serious problems than that of a cantilever proper; on the contrary, the final erection adjustments are simpler in the case of the arch.

Two Comparative Designs of Arch Type.

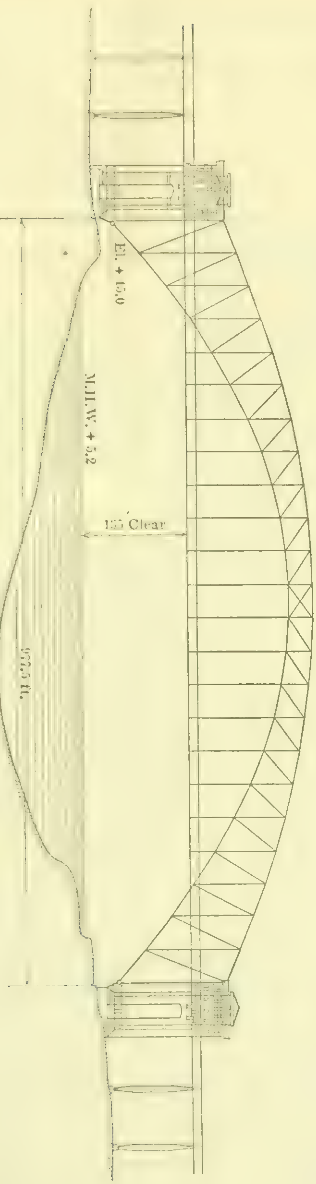
Two designs for an arch bridge were made by Mr. Lindenthal in 1905. Fig. 6 represents the two-hinged crescent arch, as used, for instance, for the railroad bridges over the Garabit Valley, in France, and over the Douro River, in Portugal, with spans of 541 and 525 ft., respectively. Fig. 7 represents the two-hinged, spandrel-braced arch, similar to the bridges over the Rhine at Düsseldorf and Bonn, which have spans of 595 and 614 ft., respectively. The two designs were made for the same loads and specifications as the cantilever or suspension designs, but ordinary carbon steel was assumed throughout, because, at prevailing prices, nickel steel ($40 per ton higher than carbon steel) did not seem to offer any saving. The estimated weight of steelwork was slightly in favor of the crescent arch, but the spandrel-braced arch offered greater advantages for the cantilever erection. Although both designs are pleasing in appearance, the spandrel arch, owing to its height increasing from the center toward the ends, is more expressive of rigidity than the crescent arch, the ends of which appear to be unnaturally slim in comparison with the great height at the center.

The three-hinged arch was not considered. It is not cheaper than the two-hinged arch, and is not as rigid. The fact that the two-hinged arch is statically indeterminate is frequently cited as an objection to that type. This is not justified. There is no more uncertainty in the stress distribution in that type than in a so-called statically determinate structure with riveted connections, and even pin connections do not remove the uncertainty, as is now well recognized. Moreover, if desired, it is always possible, as has been done in the case of the Hell Gate Bridge, to erect the two-hinged arch so that it is statically determinate for dead load. This, however, is a convenience in erection rather than an advantage as regards stress action. Of course, the above characteristic of the two-hinged arch is a serious objection where no solid foundations can be obtained, because of the uncertainty of



CRESCENT ARCH DESIGN (1905)

Fig. 6.



SPANDREL BRACED ARCH DESIGN (1905)

Fig. 7.

stresses which may be produced by a spreading of the foundations. A two-hinged arch required unyielding abutments.

Modifications in Adopted Arch Design.

The spandrel-braced arch type was finally adopted. Before the design was worked out in detail, the top chord was changed by giving it a slight reversal of curve toward the ends, in order to provide for a stout portal and wind bracing for the top chords, and also improve the silhouette of the arch. The towers were increased in height, and their architectural features modified.

In 1907, the design of the tower shown in Fig. 8 was submitted to the Municipal Art Commission of New York. This Commission, although not objecting to the design as a whole, disapproved of the decorative features of the towers and their bases. The towers then received several further modifications until, in 1912, shortly before construction started, the design illustrated in Fig. 9 was finally adopted. The tower in this design represents a great improvement over that shown in Fig. 8, being more impressive in outline and simpler in architectural ornament and, therefore, more in harmony with the simple, imposing forms and lines of the bridge proper.

It should also be mentioned that, in 1910, the steel superstructure was re-designed to carry Cooper's E-60 loading and a solid ballasted floor. This loading had already been adopted by a number of railroads, including the New York, New Haven and Hartford Railroad, which is to use the bridge. High-carbon steel was adopted in place of the ordinary structural steel. This last design, moreover, was based on special rules of design prepared by Mr. Lindenthal, as abstracted hereinafter. These resulted in a heavier floor system, a stronger connection, and heavier details throughout. Four lines of stringers and floor-beam brackets, strong enough to carry trolley traffic, were provided outside of the trusses. These modifications increased the total weight of the steelwork to 18 900 tons.

4.—General Arrangement, Proportions, and Cost of Arch Bridge.

The Hell Gate Bridge, as built (Plate XVII), is a two-hinged spandrel-braced arch, carrying four tracks. Its general proportions were dictated partly by local conditions and partly by the requirements for economy and rigidity. The artistic outlines of the steel

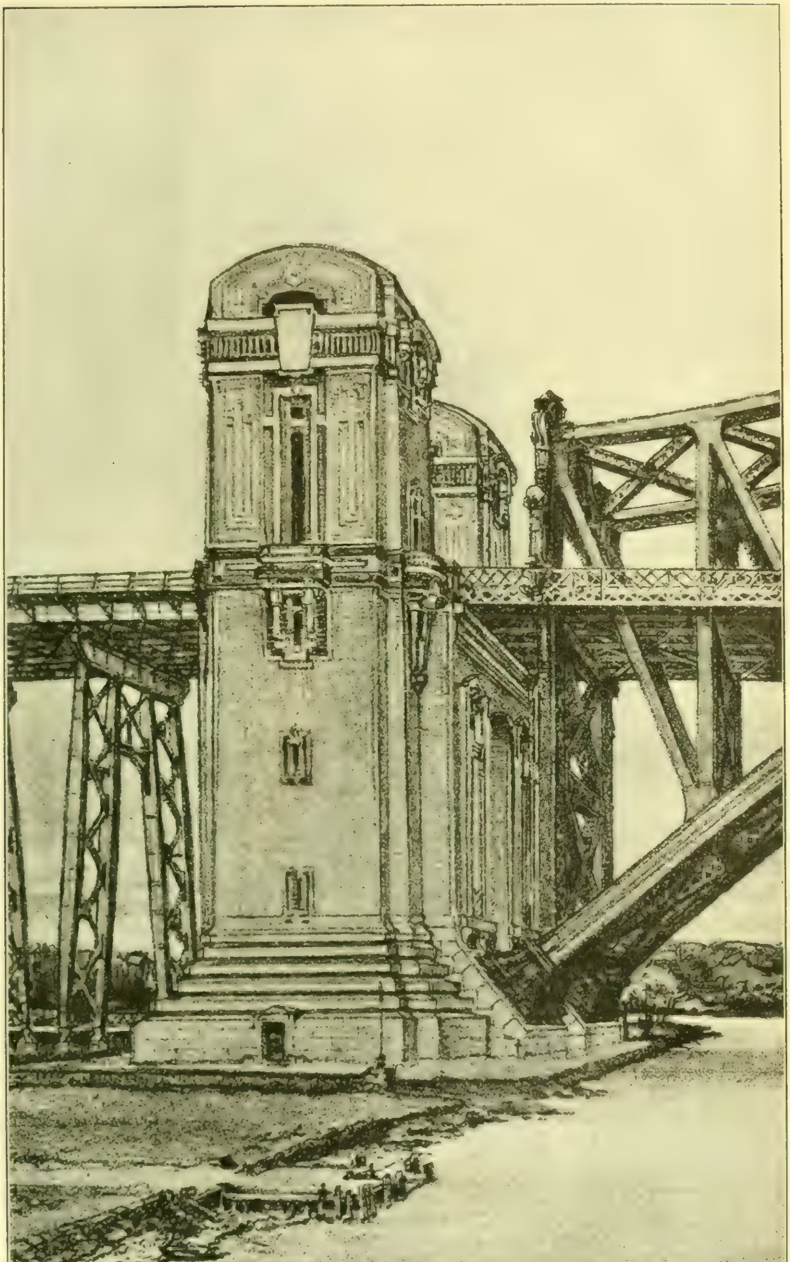


FIG. 8.—PERSPECTIVE VIEW OF ARCH DESIGN (1906), HELL GATE BRIDGE.

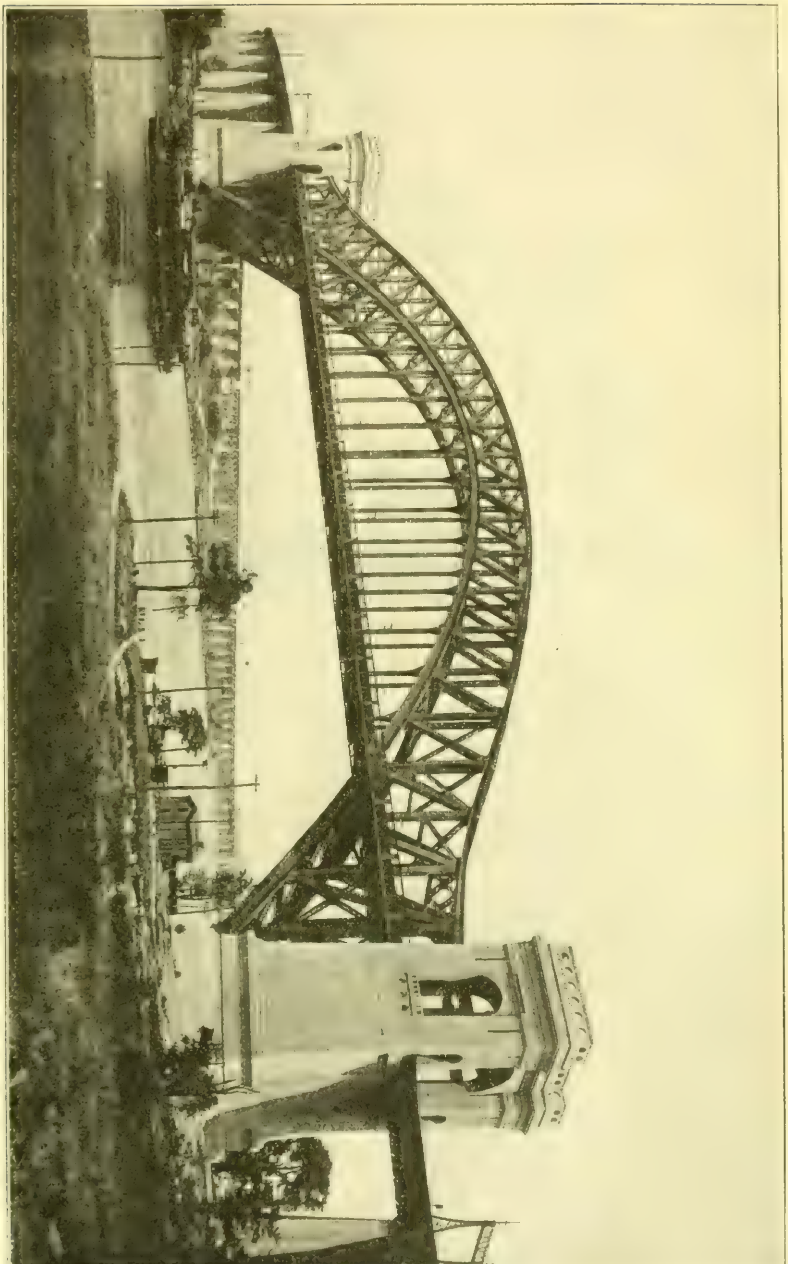
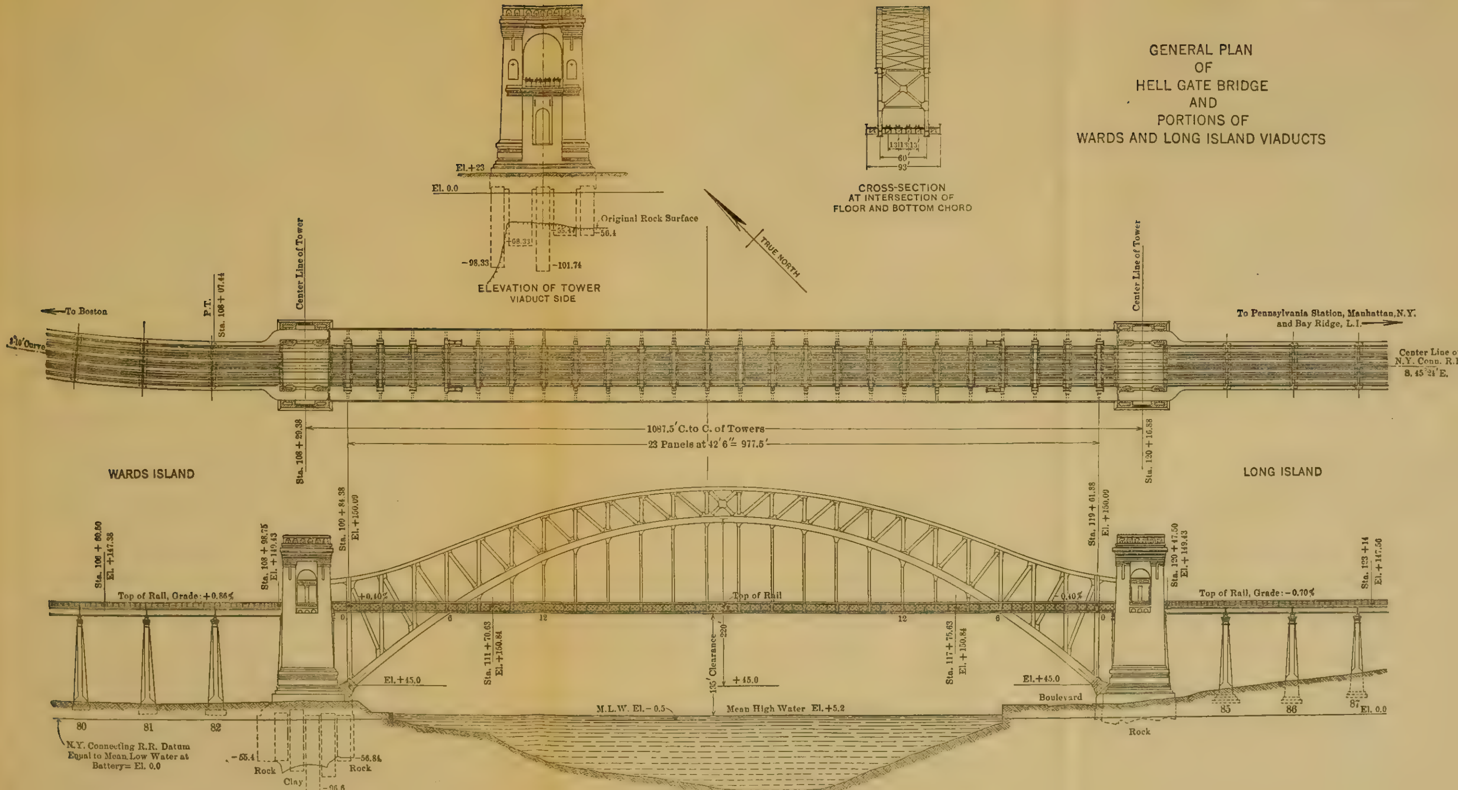


Fig. 9.—Perspective View of Final Design (1912), Hell Gate Bridge.

PLATE XVII.
PAPERS, AM. SOC. C. E.
OCTOBER, 1917.
AMMANN ON
HELL GATE BRIDGE.

GENERAL PLAN OF HELL GATE BRIDGE AND PORTIONS OF WARDS AND LONG ISLAND VIADUCTS



superstructure are the result of the proper interpretation of the economic and engineering requirements of the structure.

Span Length and Rise[*] *of Arch.*—The span length is 1 017 ft. between faces of masonry towers at track level, 995 ft. 1$\frac{3}{8}$ in. between centers of bearings on skewbacks, and 977 ft. 6 in. between centers of hinges. The length was determined indirectly by the clear height of 135 ft. above mean high water, as prescribed by the War Department, and which had to be maintained for a width of about 700 ft. between the established bulkhead lines. The upper corners of this clearance rectangle fixed the intersection points of the floor with the bottom chord, and thus, in combination with the chosen rise and form of arch, fixed the location of the abutments. The Long Island abutment had to be placed so as to clear the "boulevard" which runs along the shore line.

The exact rise of the center line of the bottom chord is 228 ft. 9$\frac{11}{16}$ in. above the centers of bearings on skewbacks, and 220 ft. 0 in. above the centers of hinges, which gives a ratio of rise to span length of 1 : 4.5. This ratio is about the most economical under the given conditions. The weight of an arch of this type and length varies inappreciably for variations in rise between one-sixth and one-fourth of the span, the absolute minimum being probably nearer the lower value. The greater rise in this case secured greater economy, because, for the given length between the intersection points of the lower chord and the floor, a flatter arch would have required a longer span and, therefore, more metal and more expensive abutments and foundations.

Shape of Bottom Chord.—The panel points of the bottom chord lie on a parabola, this being the line of equilibrium of the bottom chord as an independent arch rib, when covered with a uniform load over the whole span. To approach this condition, as nearly as possible, and thus secure for each truss a massive arch rib expressive of great strength, and transmit the loads in the most direct way to the abutments, the trusses were designed and erected to act as three-hinged arches for the entire weight of steelwork, the joint at one of the center panel points of the bottom chord acting as the middle hinge. This arrangement is conducive to economy and rigidity.

Height of Arch Trusses, and Shape of Top Chord.—The total height of the arch trusses from center of hinge to center of top chord at the middle of the span is 260 ft. 2$\frac{5}{8}$ in., and the total height of the steel

superstructure above mean high water is 305 ft. The height of the trusses at the quarter points of the span, where the greatest deflections occur, was chosen at 60 ft., or slightly more than one-fourth of the rise of the bottom chord. This proportion of height to rise insures ample rigidity. Besides, it is sufficient to keep the maximum live-load stresses in the bottom chord approximately within the stresses from live load covering the whole span, and, therefore, no extra material is required in the bottom chord for the otherwise large stresses from partial live load.

On the other hand, for a greater proportion of height to rise than that chosen, the aggregate weight of the top chord and web members increases, and the weight of the bottom chord remains nearly constant.

The height of the trusses was decreased toward the center to 40 ft. $2\frac{5}{8}$ in., or approximately one-twenty-fourth of the span length, so as to reduce the temperature stresses, which are greatest at the center. The height of 140 ft. at the ends of the trusses was determined by the necessity for rigid portals between the end posts above the track floor. These assumptions for height resulted in the slightly reversed curve of the top chord, which produces a very pleasing sky line.

Width.—The width of 60 ft. between centers of trusses resulted from the required clear width of 53 ft. for the four tracks (the distance between centers of tracks being 13 ft.) and an allowance of 7 ft. for the width of the bottom chord at its intersection with the floor. This width, being one-sixteenth of the span length, was sufficient for lateral stability and rigidity, and therefore it was not necessary to spread the trusses or to place them in inclined planes. To obtain great lateral rigidity of the suspended floor and an economical floor wind truss, however, the latter is made 93 ft. wide, its chords being placed $16\frac{1}{2}$ ft. outside of the main trusses and carried by cantilever extensions of the floor-beams.

Web System and Panel Length.—The web system consists of a single line of verticals and diagonals, the latter falling toward the center of the span, as commonly used in arch bridges of this type. The system is simple, economical, and free from large secondary stresses. There are twenty-three equal panels, each $42\frac{1}{2}$ ft. long. For a two-hinged arch, an odd number of panels produces a better appearance than an even number. The panel length was chosen with a view to obtain the most economical floor and truss web system. The latter was secured by

making the average inclination of the diagonals about 45° with the horizontal.

General Arrangement of Arch Bracing.—The transverse bracing between the two trusses comprises a lateral system along the top chords, a lateral system along the bottom chords, and sway-frames and portals in the planes of the first five verticals at each end of the span. Sway-frames between the other truss verticals and between the floor suspenders have been omitted purposely, as they are not needed and would have to be very heavy to resist the stresses from unequal deflection of the two trusses under one-sided load. The top lateral truss is assumed to transmit its reactions to the portals between the end posts and through these and the sway-frames below the floor to the bearings. The wind forces which act along the bottom chords are transmitted through the bottom lateral truss directly to the arch bearings. Owing to the polygonal shape of the chords, components of the lateral stresses are transmitted into the main trusses at each panel point, which, although small, had to be considered in proportioning the truss members. At the intersections of the floor with the bottom chord, the lateral bracing between these chords had to be interrupted to provide the necessary head-room above the floor. Stiff portals were substituted for the laterals at these points, and the chords were proportioned for the additional bending stresses.

General Arrangement of Floor System.—The floor system (Plates XX and XXIV) comprises the following parts:

1.—A floor-beam at each panel point, rigidly framed into the trusses at the first four verticals at each end of the span and hung from the trusses by suspenders in the middle portion of the span.

2.—Eight lines of railroad stringers, 6 ft. 6 in. apart, framed into the floor-beams and braced together in pairs for each track by top and bottom laterals and sway-frames. Each pair carries a concrete trough which supports the ballasted track.

3.—Four lines of sidewalk stringers, one pair outside of each track, framed into cantilever extensions of the floor-beams. These stringers support only a light sidewalk, but are made strong enough to carry the trolley line which was contemplated.

4.—Two lines of lattice girders, one on each side of the floor, placed 16½ ft. outside of the center line of the main truss, and carried at the end of the floor-beam extensions. The function of these girders is to

screen the floor system and thus secure a more uniform and neat appearance. At the same time, these girders act as railings, and their bottom chords form the chords of the floor lateral truss.

5.—A floor lateral truss to resist the wind and lateral forces which act on the trains and floor.

6.—Two "braking girders", one at each intersection of the floor with the main trusses. These girders transmit the longitudinal forces from braking and traction from the stringers to the main trusses, and thus eliminate serious horizontal bending of the floor-beams.

Provision for Expansion of Floor.—The floor had to have at least one expansion joint at or between its intersections with the bottom chord (Panel Points 6, Plates XVII and XX), so as not to be strained by temperature changes or deformation of the arch trusses.

The expansion of the floor for a change in temperature of $\pm$ 72° Fahr. is $\pm$ 4.1 in., but this is partly offset by an increase of $\pm$ 1.6 in. in the distance between Points 6, due to the temperature deformation of the arch truss, leaving a movement from the normal position, of $\pm 4.1 \mp 1.6 = \pm 2.5$ in., which had to be provided for at the expansion joint. The effect of a maximum live load covering the entire span is to open the joint by 0.1 in., which is negligible.

In deciding on the location of the expansion joint, the following conditions had to be taken into consideration:

First.—To secure the greatest lateral rigidity, the floor lateral system should be such as to cause the least lateral deflections.

Second.—To avoid large stresses in the stringers and their connections from the longitudinal force, the distance between the expansion joint and the braking girder should be as small as possible.

Third.—The floor suspenders should be subject to the least possible bending in the plane of the truss from the expansion of the floor.

To meet all these conditions, the expansion joint was placed at Panel Point 12, six panels from the Wards Island end. At the corresponding Point 12 on the Long Island side, the floor laterals are rigidly connected at the center of the floor-beam, but the wind chords are cut, so as to secure hinge action of the floor-lateral truss. The latter, therefore, forms a three-span cantilever truss with a suspended span between Points 12, Cantilever Arms 12—6, and Anchor Arms 6—0.

The suspended span delivers its reactions to the cantilever arms by a fixed connection of the diagonals at the center of Floor-beam 12 (Long Island side) and a sliding bearing at the center of Floor-beam 12 (Wards Island side). The reactions at Points 6 are transmitted to the bottom chord lateral system and, through this, to the arch bearings. The reactions of the floor-lateral truss at the ends are transmitted to the sway-frames between the end posts and, through these, to the bearings.

The longitudinal force from Part 0—12 (Wards Island side) is transmitted to the braking girder at 6 (Wards Island side), and the force from Part 12 (Wards Island side) to 0 (Long Island side) is transmitted to the braking girder at 6 (Long Island side). The stringer connections to the floor-beams are made strong enough to resist safely the longitudinal force in addition to the vertical shear.

Cost.—The Hell Gate Bridge contains approximately 110 000 cu. yd. of masonry in the towers and foundations and 19 400 tons of steel in the steel superstructure. (Detailed quantities are given under the respective headings.)

The cost of construction is approximately as follows:

Towers and foundations	$1 700 000
Steel superstructure	1 900 000
Concrete flooring and tracks	100 000
Total	$3 700 000

5.—Masonry Towers and Foundations.

The massive masonry towers which flank the steel arch greatly enhance the appearance of the bridge and give it its monumental character. They also give expression to the solidity of the abutments to resist the great thrust of the arch. Without the towers, the statically trained eye would want that expression of stability, because of the comparative flatness of the shores.

This static requirement, however, is not merely an apparent one. Preliminary wash-borings indicated that the foundations had to go to considerable depth, at least on the Wards Island side. There having been doubt as to the reliability of the wash-borings, the depth of rock was established later by core-borings at from 55 to 140 ft. below mean high water line. To restrict the size of the foundation to a minimum,

it was necessary to provide above the ground a mass of masonry, the weight of which, combined with the inclined reaction of the arch, would give a steep resultant, passing well within the middle third of the foundation area, so that the edge pressure could be kept within permissible limits.

To be expressive of their purpose, the towers were designed architecturally as massive masonry blocks with simple outlines and plain structural ornamentation (Fig. 10). In working out the architectural form and details of the towers, Mr. Lindenthal had the valuable assistance and advice of Mr. Henry Hornbostel as Consulting Architect.

Towers Above Foundations.—Plate XVIII shows the type of construction of the towers above the foundations. The dimensions of the towers are 103 by 139 ft. at the ground level and diminish along parabolic lines to 61 by 105 ft. at the top. The total height above ground is 220 ft., and the extreme height above bottom of foundation is 345 ft. Each tower has a solid base, which acts as an abutment for the arch bearings and distributes the pressure over the foundations. On the Long Island side, the base, with the foundation course, forms a monolithic slab, 140 by 104 ft., with an average thickness of 49 ft., and rests on gneiss bed-rock, which was encountered at from 15 to 38 ft. below the surface, and was reached by open excavation. The maximum foundation pressure is 8½ tons per sq. ft. On the Wards Island side, the base is 140 by 119 ft. and 40 ft. thick, and rests on the caisson foundation described subsequently.

Above this base, and up to the track floor, the towers are of hollow cellular construction, consisting of the four exterior walls and three interior walls parallel to the tracks. Above the track floor, the transverse walls are pierced by a main arch over the four tracks, and two smaller side arches over the footwalks. The longitudinal walls have also an arch opening for architectural reasons. The towers are topped by a flat roof surrounded by an ornamental balustrade. Stairs lead from an entrance at the ground level through the base and interior vaults to the track floor and roof, and also to the ends of the top chords of the steel arch. The towers are of concrete with granite facing. The concrete is well reinforced with vertical and horizontal steel rods in order to prevent temperature and shrinkage cracks. The track floor and roof are heavily reinforced with steel girders. The Snare and

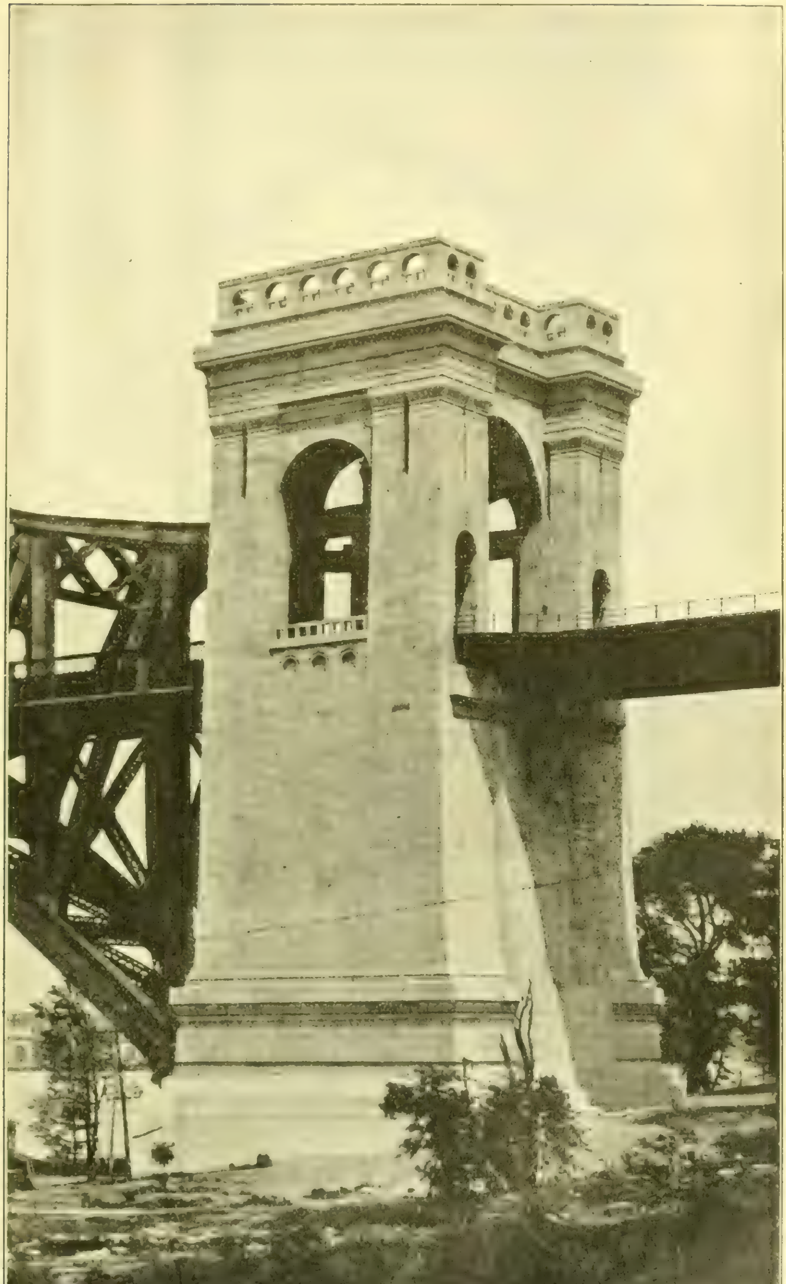
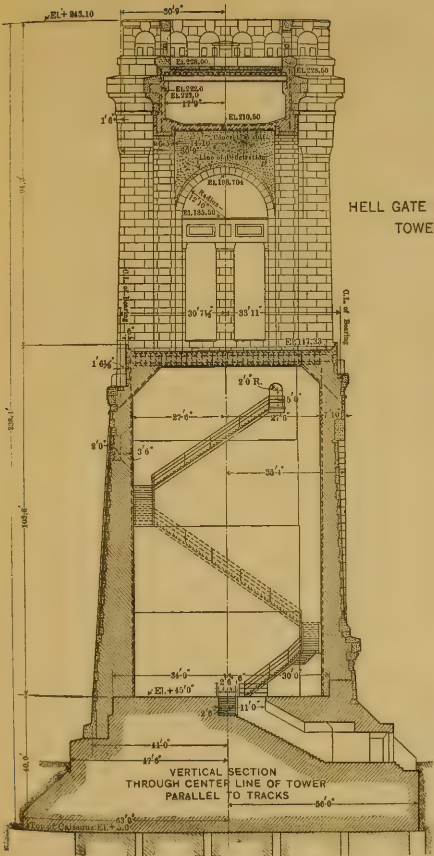


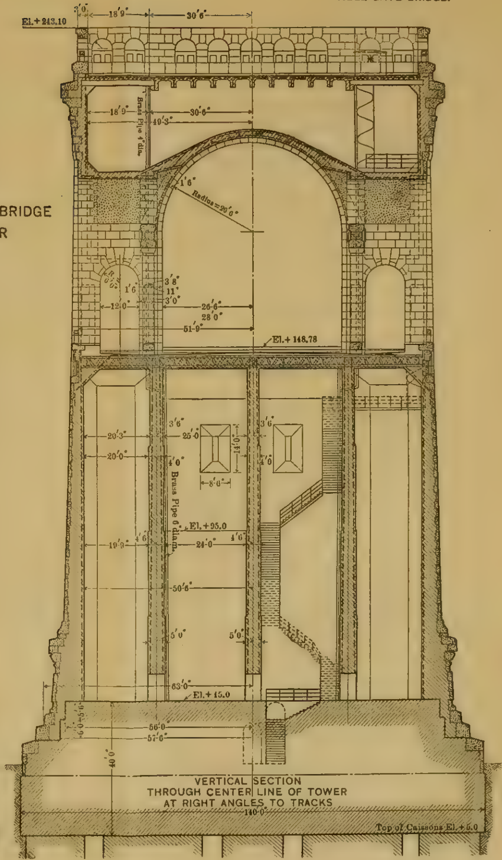
FIG. 10.—LONG ISLAND TOWER OF HELL GATE BRIDGE.

PLATE XVIII.
PAPERS, AM. SOC. C. E.
OCTOBER, 1917.
AMMANN ON
HELL GATE BRIDGE.

HELL GATE BRIDGE TOWER



VERTICAL SECTION THROUGH CENTER LINE OF TOWER PARALLEL TO TRACKS



VERTICAL SECTION THROUGH CENTER LINE OF TOWER AT RIGHT ANGLES TO TRACKS

Triest Company was the contractor for the towers above the bases, and the Ryan Construction Corporation for the bases.

Wards Island Tower Foundation.—On the Wards Island side, the tower base rests on twenty-one concrete caissons, all sunk by the pneumatic process to depths varying from 37 to 107 ft. below mean high water, which is 20 ft. below the ground surface. (Plate XIX.) The caissons are arranged in five rows parallel to the axis of the bridge. Each outer row and the middle one consists of five cylindrical caissons, 18 ft. in diameter, which are calculated to carry only vertical pressure. Each of the two other rows consists of three rectangular caissons, 30 by 41 ft., which are interlocked by concrete keys extending nearly the full depth of the caissons. These two rows of caissons thus form two rectangular blocks, 30 by 125 ft., which are calculated to resist entirely the horizontal pressure from the arch. They exert a maximum edge pressure on the rock foundation of 20 tons per sq. ft., if skin friction and buoyancy are neglected. The space for the keys was excavated and filled with concrete, partly with and partly without the use of air pressure, after the caissons had been sunk to their final depth.

The dissection of the foundation into twenty-one individual caissons was advisable on account of the large size of the tower base and the greatly varying depth to solid rock. The formation of the bed-rock below Wards Island is very peculiar. One of the lines of cleavage between the dolomitic limestone formation and the gneiss rock which run parallel with the East River, appears to pass right under the Wards Island tower. About 45% of the foundation on the river side is on limestone and about 25% on the land side is on gneiss. Between these two rock formations there is a crevasse of unknown depth and of from 15 to 60 ft. width. Its sides are almost vertical, corresponding to the vertical stratification of the rock. The crevasse is filled mostly with red clay and some boulders. The clay is practically impervious to water, which is shown by the fact that the excavation was in part carried on with an air pressure of only 18 lb. at a depth of 100 ft. below water level, and, for some caissons, to 18 ft. below the cutting edge. In its natural state, the clay is very hard and has a high bearing capacity, but, in water, it dissolves readily.

Three of the fifteen cylinder caissons rest entirely on this clay, at depths of from 94 to 123 ft. below the surface, where there is no danger of disturbance. Under the rectangular caissons, the crevasse was

bridged over by a concrete arch which was built at considerable risk below the cutting edge of the caissons. For this purpose, the caissons were supported temporarily by pilasters of rubble masonry built down to the intrados of the arch previous to the excavation for the latter (Plate XIX). All caissons are of 1:2:4 concrete, well reinforced horizontally and vertically with steel rods. Details of the working chambers and cutting edge are shown in Fig. 11.

Owing to the unusual and difficult character of the Wards Island foundation, the Company decided to do the work with its own forces, under the direction of the Chief Engineer. P. G. Brown, M. Am. Soc. C. E., as Managing Engineer, was in immediate charge of this work.

Quantities.—The two towers contain approximately 110 000 cu. yd. of masonry, of which 28 000 cu. yd. are in the Wards Island foundation. The principal quantities are:

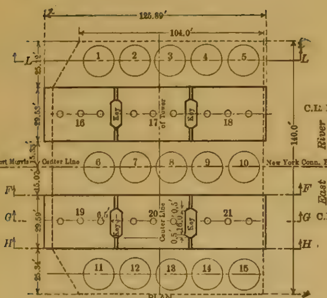
Concrete (1:2:4 and 1:2½:5)......	99 000 cu. yd.
Granite	11 000 cu. yd.
Steel reinforcement	1 000 tons
Structural steel	500 tons

6.—Details of Design, and Weight of Steel Superstructure of Hell Gate Bridge.

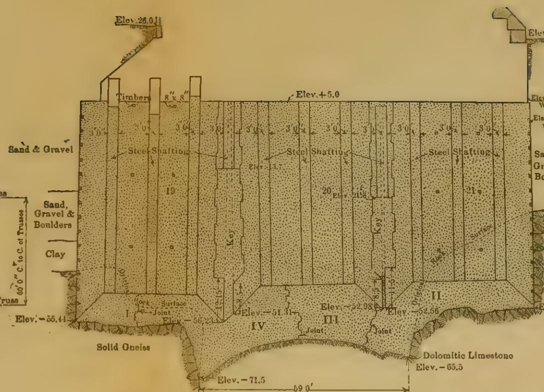
The details of design of the steel superstructure have been worked out to conform primarily to the requirements for strength and rigidity and next for neat appearance, without extra expense for structural ornamentation. Stress sheets and complete detailed plans were prepared by the Consulting Engineer, on which the Contractor was required to base his working drawings, and he was therein given opportunity to utilize his experience in fabrication, erection, special devices, and working methods. This is the proper procedure in the case of a large bridge in which many details are of unusual dimensions and composition.

Sections of Truss Members.—The make-up of the sections of the truss members was largely governed by the necessity for riveted connections. Pin connections were not considered. They are objectionable in members subject to reversal of stress, as they impair the rigidity and durability of the bridge. Because of the riveted connections, the number of webs of all members was limited to two. Fig. 12 shows the typical sections of the various truss members.

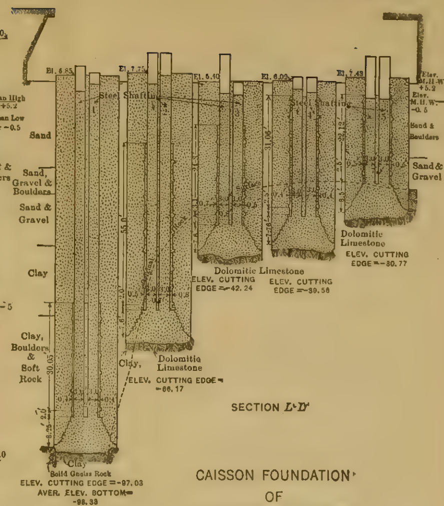
PLATE XIX.
PAPERS, AM. SOC. C. E.
OCTOBER, 1917.
AMMANN ON
HELL GATE BRIDGE.



PLAN
SHOWING DIMENSIONS AND
LOCATION OF CAISSONS AT ELEVATION +5.0

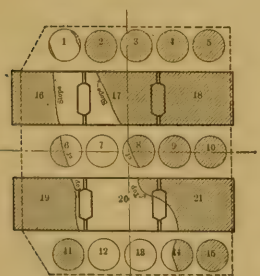


SECTION G-G

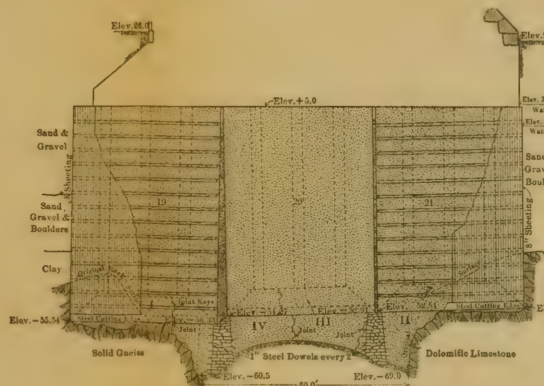


SECTION L-L

CAISSON FOUNDATION
OF
WARDS ISLAND TOWER



Shaded portion represents Rock
PLAN SHOWING LOCATION OF ROCK
AT ELEVATION OF CUTTING EDGES



SECTION H-H

NOTES:

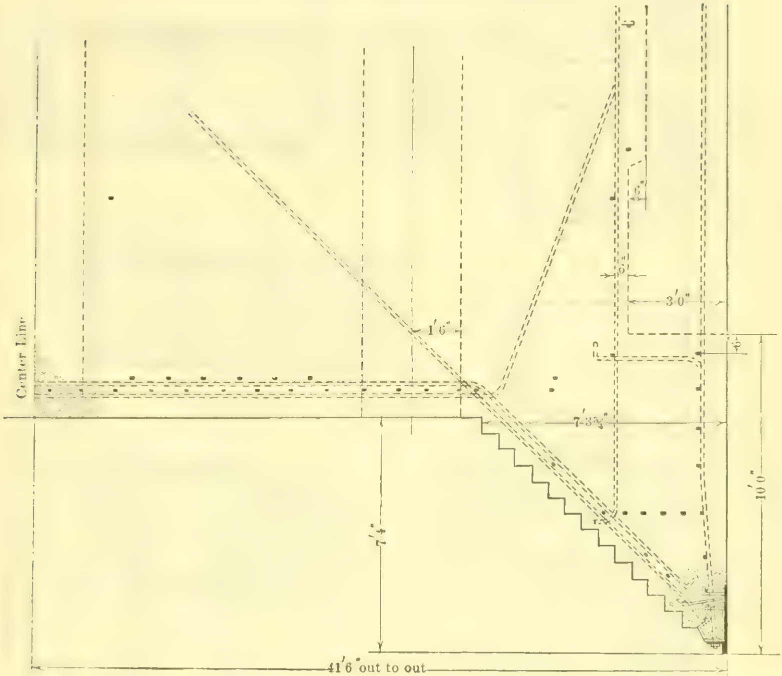
Caissons shown on this plate, were sunk in the following order: 20 19-21
Rubble masonry piers built under E. & W. Cutting edges of Caissons 19 & 20 at N. & S. sides and under 20 & 21 at S. side only. These piers were constructed to support caissons during excavation for Arch, and were left in place.

Order of procedure in excavating and concreting working chamber.

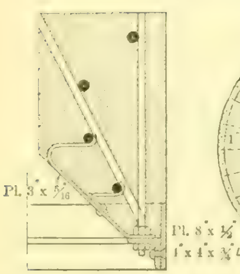
1st–	Part	I	excavated and concreted continuously
2nd–	"	II	" " " "
3rd–	"	III	" " " "
4th	"	IV	" " " "

Keyways between caissons 19 & 20 and 20 & 21 excavated and sealed under air pressure.
Total linear feet of sinkage = 171′
Steel shafting left in place.
Average air pressure during arch construction = 30 lb.
" " " " sinking = 30 lb.
Concrete in all Caissons: 1 part Portland Cement, 3 parts Sand, 4 parts Broken Stone. (Large Rubble Stone embedded in concrete)
Datum: All elevations given, refer to datum of New York Connecting R.R., which is equal to Mean Low Water at Battery = Elev. 0.0

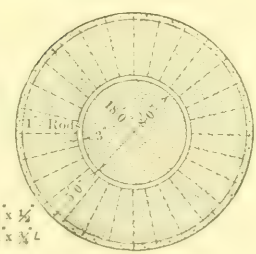
DETAILS OF WORKING CHAMBER OF
RECTANGULAR CAISSONS
WARDS ISLAND TOWER FOUNDATION.



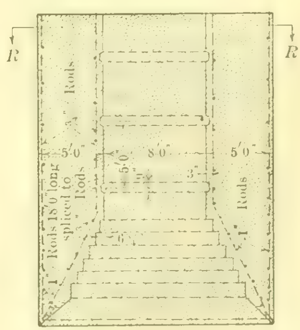
DETAILS OF WORKING CHAMBER OF
CYLINDRICAL CAISSONS
WARDS ISLAND TOWER FOUNDATION.



DETAIL OF CUTTING EDGE



SECTION R-R



SECTION ON CENTER LINE

FIG. 11.

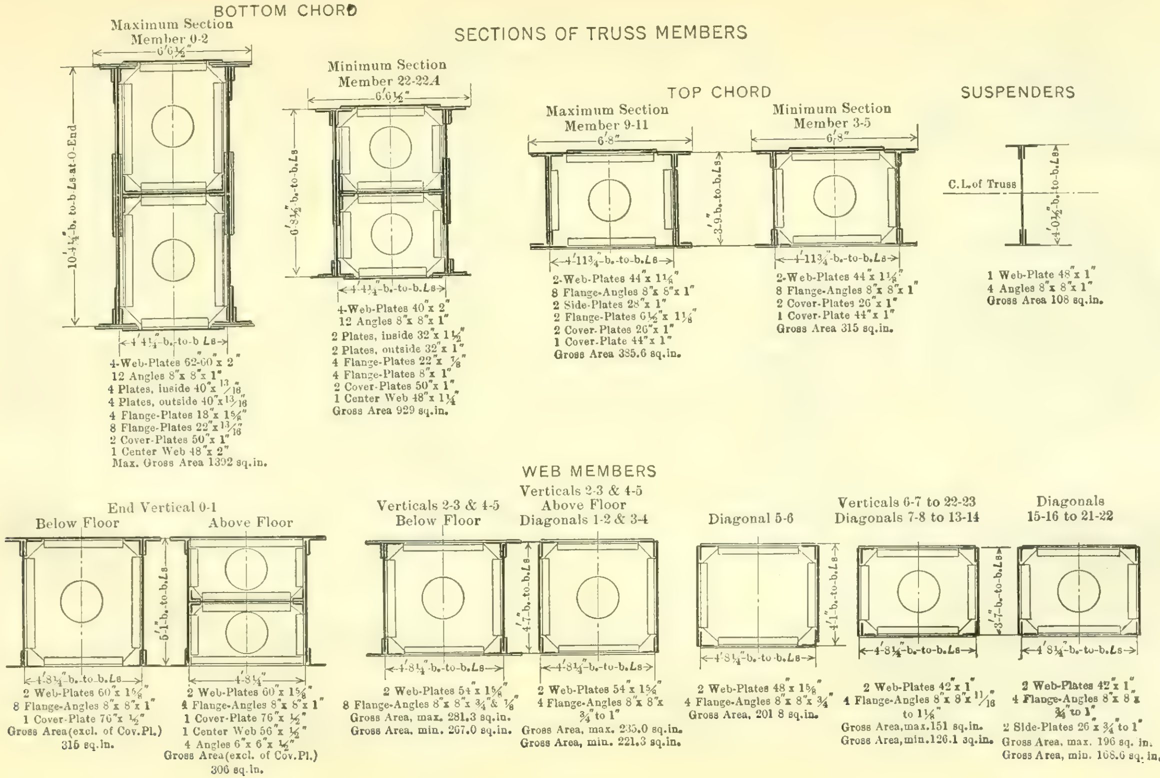

SECTIONS OF TRUSS MEMBERS
BOTTOM CHORD
Maximum Section Member 0-2
6'6½"
10'4¼"-b. to-b-Ls-at-O-End
4'4¼"-b.-to-b Ls
4-Web-Plates 62-60" x 2"
12 Angles 8"x 8"x 1"
4 Plates, inside 40"x 13/16"
4 Plates, outside 40"x 13/16"
4 Flange-Plates 18"x 1⅝"
8 Flange-Plates 22"x 13/16"
2 Cover-Plates 50"x 1"
1 Center Web 48"x 2"
Max. Gross Area 1392 sq.in.
Minimum Section Member 22-22A
6'6½"
6'8½"-b.-to-b.Ls
4'4¼"-b.-to-b.Ls
4-Web-Plates 40"x 2"
12 Angles 8"x 8"x 1"
2 Plates, inside 32"x 1½"
2 Plates, outside 32"x 1"
4 Flange-Plates 22"x ⅞"
4 Flange-Plates 8"x 1"
2 Cover-Plates 50"x 1"
1 Center Web 48"x 1¼"
Gross Area 929 sq.in.
TOP CHORD
Maximum Section Member 9-11
6'8"
4'11¾"-b.-to-b.Ls
2-Web-Plates 44"x 1⅛"
8 Flange-Angles 8"x 8"x 1"
2 Side-Plates 28"x 1"
2 Flange-Plates 6½"x 1⅛"
2 Cover-Plates 26"x 1"
1 Cover-Plate 44"x 1"
Gross Area 385.6 sq.in.
3'9"-b.-to-b.Ls
Minimum Section Member 3-5
6'8"
4'11¾"-b.-to-b.Ls
2-Web-Plates 44"x 1⅛"
8 Flange-Angles 8"x 8"x 1"
2 Cover-Plates 26"x 1"
1 Cover-Plate 44"x 1"
Gross Area 315 sq.in.
SUSPENDERS
C.L.of Truss
4'0½"-b.-to-b.Ls
1 Web-Plate 48"x 1"
4 Angles 8"x 8"x 1"
Gross Area 108 sq.in.
WEB MEMBERS
End Vertical 0-1
Below Floor
4'8¼"-b.-to-b.Ls
2 Web-Plates 60"x 1⅝"
8 Flange-Angles 8"x 8"x 1"
1 Cover-Plate 76"x ½"
Gross Area(excl. of Cov.Pl.)
316 sq.in.
5'1"-b.-to-b.Ls
Above Floor
4'8¼"
2 Web-Plates 60"x 1⅝"
4 Flange-Angles 8"x 8"x 1"
1 Cover-Plate 76"x ½"
1 Center Web 56"x ½"
4 Angles 6"x 6"x ½"
Gross Area(excl. of Cov.Pl.)
306 sq.in.
Verticals 2-3 & 4-5
Below Floor
4'8¼"-b.-to-b.Ls
2 Web-Plates 54"x 1⅝"
8 Flange-Angles 8"x 8"x ¾"& ⅞"
Gross Area, max. 281.3 sq.in.
Gross Area, min. 267.0 sq.in.
4'7"-b.-to-b.Ls
Verticals 2-3 & 4-5
Above Floor
Diagonals 1-2 & 3-4
4'8¼"-b.-to-b.Ls
2 Web-Plates 54"x 1⅝"
4 Flange-Angles 8"x 8"x
¾" to 1"
Gross Area, max. 235.0 sq.in.
Gross Area, min. 221.3 sq.in.
Diagonal 5-6
4'1"-b.-to-b.Ls
4'8¼"-b.-to-b.Ls
2 Web-Plates 48"x 1⅝"
4 Flange-Angles 8"x 8"x ¾"
Gross Area, 201 8 sq.in.
Verticals 6-7 to 22-23
Diagonals 7-8 to 13-14
4'8¼"-b.-to-b.Ls
3'7"-b.-to-b.Ls
2 Web-Plates 42"x 1"
4 Flange-Angles 8"x 8"x 11/16"
to 1⅛"
Gross Area,max.151 sq.in.
Gross Area,min.126.1 sq.in.
Diagonals
15-16 to 21-22
4'8¼"-b.-to-b.Ls
2 Web-Plates 42"x 1"
4 Flange-Angles 8"x 8"x
¾" to 1"
2 Side-Plates 26"x ¾" to 1"
Gross Area, max. 196 sq. in.
Gross Area, min. 168.6 sq. in.

FIG. 12.

Bottom Chord Section.—The bottom chord has a closed double-box section, consisting of two vertical webs, top and bottom covers, and a solid horizontal diaphragm along the center line of the chord. The effective gross area varies from 929 sq. in. at the crown to 1 392 sq. in. at the bearings. With the exception of the bottom chord of the new Quebec Bridge, of 1 800 ft. span, which has a maximum section of 1 902 sq. in., the Hell Gate Bridge has the largest chord section so far built. The width of the chord is 6 ft. 6½ in. throughout.

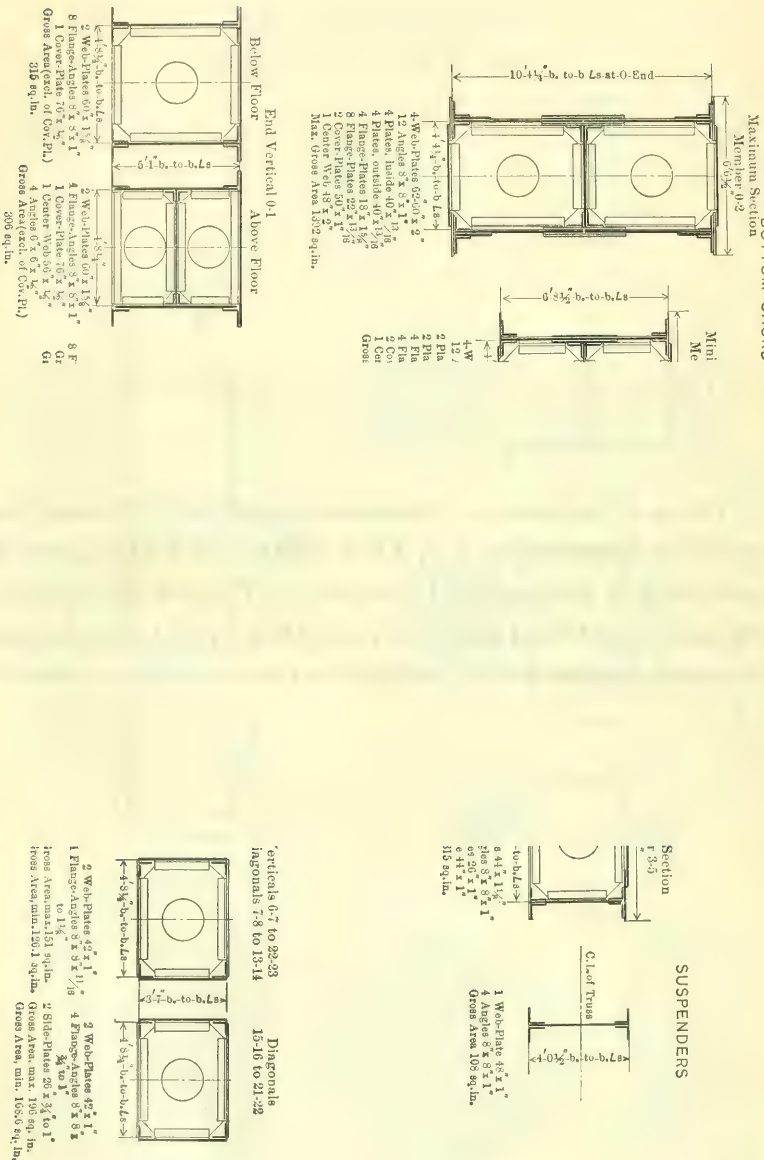
The depth, from out to out of covers, increases gradually from 7 ft. 0¼ in. at the crown to a maximum of 10 ft. 9¾ in. at the bearings, the greatest depth over all being 11 ft. 4¼ in. In that way the thickness of web was kept uniform, and the bottom chord, as the carrying mem-

Owing to the number of illustrations in this paper, it has not been possible to prepare Plates XX, XXII, XXIX, and XXXI in time for publication in this number of *Proceedings.* They will be added when the paper is published finally in *Transactions;* and, before the paper is presented, copies will be available for those who wish to discuss it.

is superior to sections made up of two or more webs connected by latticing and tie-plates, but it is adapted only to heavy chords or posts of large bridges, for which it can be made of sufficient dimensions to allow access to the inside for the purpose of riveting, inspection, and painting.

A further unusual feature of these chords is that their main webs, and the web of the center diaphragm, are single plates of the extraordinary thickness of 2 in.

Large compression members are undoubtedly stronger when made up of single thick plates than of several thin plates tack-riveted together; besides, many rivets are saved by using single thick plates. It is frequently maintained that better material is obtained in the


BOTTOM CHORD
Maximum Section
Member 0-2
6'0½"
10'-4¼"-b. to-b Ls-at-0-End
4'4¼"-b.-to-b Ls
4-Web-Plates 62-60" x 2"
12 Angles 8" x 8" x 1"
4 Plates, inside 40" x 13/16"
4 Plates, outside 40" x 13/16"
4 Flange-Plates 18" x 1⅝"
8 Flange-Plates 22" x 13/16"
2 Cover-Plates 50" x 1"
1 Center Web 48" x 2"
Max. Gross Area 1392 sq.in.
6'3½"-b.-to-b.Ls
End Vertical 0-1
Below Floor
Above Floor
5'1"-b.-to-b.Ls
4'8¼"-b.-to-b.Ls
2 Web-Plates 60" x 1⅝"
8 Flange-Angles 8" x 8" x 1"
1 Cover-Plate 76" x ½"
Gross Area(excl. of Cov.Pl.)
315 sq.in.
4'8¼"
2 Web-Plates 60" x 1⅝"
4 Flange-Angles 8" x 8" x 1"
1 Cover-Plate 76" x ½"
1 Center Web 56" x ½"
4 Angles 6" x 6" x ½"
Gross Area(excl. of Cov.Pl.)
306 sq.in.
SUSPENDERS
C.L.of Truss
4'0½"-b.-to-b.Ls
1 Web-Plate 48" x 1"
4 Angles 8" x 8" x 1"
Gross Area 108 sq.in.
Diagonals
15-16 to 21-22
3'7"-b.-to-b.Ls
4'8¼"-b.-to-b.Ls
4'8¼"-b.-to-b.Ls
2 Web-Plates 42" x 1"
to 1⅛"
2 Web-Plates 42" x 1"
4 Flange-Angles 8" x 8" x
¾" to 1"
2 Side-Plates 26" x ¾" to 1"
Gross Area, max. 196 sq. in.
Gross Area, min. 168.6 sq. in.

Bottom Chord Section.—The bottom chord has a closed double-box section, consisting of two vertical webs, top and bottom covers, and a solid horizontal diaphragm along the center line of the chord. The effective gross area varies from 929 sq. in. at the crown to 1 392 sq. in. at the bearings. With the exception of the bottom chord of the new Quebec Bridge, of 1 800 ft. span, which has a maximum section of 1 902 sq. in., the Hell Gate Bridge has the largest chord section so far built. The width of the chord is 6 ft. 6½ in. throughout.

The depth, from out to out of covers, increases gradually from 7 ft. 0¼ in. at the crown to a maximum of 10 ft. 9¾ in. at the bearings, the greatest depth over all being 11 ft. 4½ in. In that way the thickness of web was kept uniform, and the bottom chord, as the carrying member, was given an expression of strength which is very satisfactory from an architectural point of view. The depth of the chord at the bearings was made as large as transportation from shop to site would permit, and, even then, special low cars were required.

Each web-plate between two panel points had to be made up of four pieces, shop-spliced longitudinally along the center line of the member and vertically at the center of the panel. To prevent distortion, each chord member is stiffened by five pairs of transverse diaphragms.

The section of the bottom chord, as previously described, marks a radical departure from usual practice. For effectiveness to resist buckling, the circular-tube section, as used in the Forth Bridge, is theoretically the best, but its fabrication is too costly for American practice. The rectangular closed-box section, if properly stiffened, is superior to sections made up of two or more webs connected by latticing and tie-plates, but it is adapted only to heavy chords or posts of large bridges, for which it can be made of sufficient dimensions to allow access to the inside for the purpose of riveting, inspection, and painting.

A further unusual feature of these chords is that their main webs, and the web of the center diaphragm, are single plates of the extraordinary thickness of 2 in.

Large compression members are undoubtedly stronger when made up of single thick plates than of several thin plates tack-riveted together; besides, many rivets are saved by using single thick plates. It is frequently maintained that better material is obtained in the

thinner than in the thicker plates. This was not substantiated by the great number of specimen tests made for the Hell Gate Bridge from material varying from ½ in. to 2 in. in thickness. In general, the thick material showed as high elastic properties and ultimate strength as the thinner material rolled from the same heat.

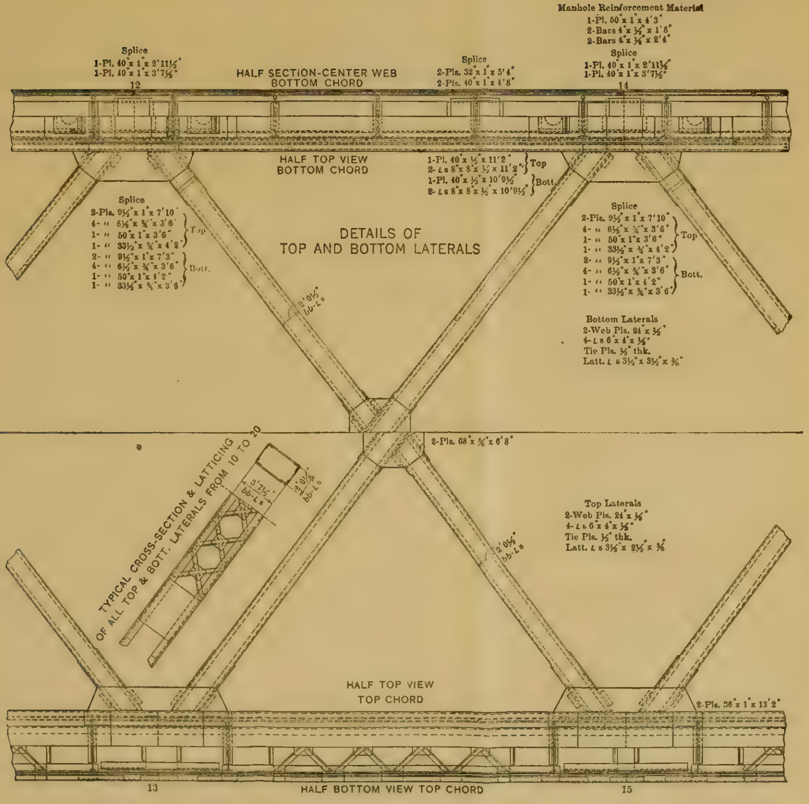
The tests* made by the Society's Special Committee on Steel Columns and Struts show a marked falling off in the compressive strength of heavy columns in comparison with light columns of approximately the same outside dimensions. It would be wrong, however, to conclude that this is due to the thicker metal in the heavy columns. It would seem to be due rather to the less efficient distribution of metal in the heavier columns. If the heavy columns had been built up of several thin plates, tack-riveted together, their compressive strength would probably have been even less. Comparative tests, to throw light on this question, would be highly desirable.

Top Chord and Web Members.—The top chord and web members have a rectangular box section with two solid webs parallel to the plane of the truss. The top chord and end posts are properly provided also with a solid cover. All open sides have stiff-angle latticing (Plates XXI and XXII). The section of the top chord ranges from 315 to 386 sq. in., and has a uniform depth of 4 ft., except in the end panel, where, for better appearance, and to allow sufficient height for entrance at the tower, the chord tapers to 5½ ft. From the end a stair leads through the chord to its top, along which two light hand-rails are provided.

The sections of the web members increase from 126 sq. in. at the crown to 315 sq. in. at the end, and the depth increases correspondingly from 42 to 60 in. A good section for compression members of moderate area is that of the two vertical posts, 2-3 and 4-5, below the floor, each flange consisting of two angles. The latticing connects only to the inside angle, but its stresses are transmitted to both angles through short tie-plates placed across the two angles (Plate XXII).

Floor Suspenders.—The suspenders, which carry the floor between its intersections with the trusses, have an I-section with a single web, 48 in. wide, placed at right angles to the plane of the truss. For appearance, and to prevent large bending stresses in the suspenders, due to the longitudinal expansion and contraction of the floor, they

* *Proceedings*, Am. Soc. C. E., for December, 1915, and December, 1916.


Manhole Reinforcement Material
1-Pl. 50"x 1"x 4'3"
2-Bars 4"x ½"x 1'8"
2-Bars 4"x ½"x 2'4"
Splice
1-Pl. 40"x 1"x 2'11½"
1-Pl. 40"x 1"x 3'7½"
12
HALF SECTION-CENTER WEB
BOTTOM CHORD
Splice
2-Pls. 32"x 1"x 5'4"
2-Pls. 40"x 1"x 4'8"
Splice
1-Pl. 40"x 1"x 2'11½"
1-Pl. 40"x 1"x 3'7½"
14
HALF TOP VIEW
BOTTOM CHORD
1-Pl. 40"x ½"x 11'2"
2-Ls 8"x 8"x ½"x 11'2" Top
1-Pl. 40"x ½"x 10'0½"
2-Ls 8"x 8"x ½"x 10'0½" Bott.
Splice
2-Pls. 9½"x 1"x 7'10"
4- " 6½"x ¾"x 3'6"
1- " 50"x 1"x 3'6"
1- " 33½"x ¾"x 4'2" Top
2- " 9½"x 1"x 7'3"
4- " 6½"x ¾"x 3'6"
1- " 50"x 1"x 4'2"
1- " 33½"x ¾"x 3'6" Bott.
DETAILS OF
TOP AND BOTTOM LATERALS
Splice
2-Pls. 9½"x 1"x 7'10"
4- " 6½"x ¾"x 3'6"
1- " 50"x 1"x 3'6"
1- " 33½"x ¾"x 4'2" Top
2- " 9½"x 1"x 7'3"
4- " 6½"x ¾"x 3'6"
1- " 50"x 1"x 4'2"
1- " 33½"x ¾"x 3'6" Bott.
2'0½" 6-Ls
Bottom Laterals
2-Web Pls. 24"x ½"
4-Ls 6"x 4"x ½"
Tie Pls. ½" thk.
Latt. Ls 3½"x 3½"x ⅜"
2-Pls. 68"x ⅝"x 6'8"
TYPICAL CROSS-SECTION & LATTICING
OF ALL TOP & BOTT. LATERALS FROM 10 TO 20
3'7½"
2'0½"
Top Laterals
2-Web Pls. 24"x ½"
4-Ls 6"x 4"x ½"
Tie Pls. ½" thk.
Latt. Ls 3½"x 2½"x ⅜"
HALF TOP VIEW
TOP CHORD
2-Pls. 38"x 1"x 13'2"
13
HALF BOTTOM VIEW TOP CHORD
15

were purposely made slender in elevation. To prevent bending stresses in the suspenders, due to the vertical deflection of the floor-beams and the horizontal deflection of the floor-lateral truss, the suspenders are connected to the floor-beams at the bottom and to the trusses at the top with 16-in. pins placed parallel to the plane of the truss (Plate XXII).

Latticing of Truss Members.—Since the failure of the Quebec Bridge in 1907, increased attention has been given to the latticing of compression members, which has led to marked improvement in this respect. Numerous tests, and a number of failures that have occurred since, have given further proof that inadequate latticing greatly reduces the buckling strength of compression members. It is now generally recognized that the latticing has a distinct static function, and bears a certain relation to the dimensions, shape, and area of section of the member.

The following simple and easily remembered rule was applied in proportioning the latticing in the Hell Gate Bridge and Approaches:

> "The latticing and its connections shall be designed to resist at any section at right angles to the axis of the member a shearing force, in pounds, at least equal to 300 times the gross area of the member, in square inches."

Stiff-angle latticing was used throughout, with a minimum thickness of ½ in., and at least two rivets for each angle. Flat lattice bars in heavy compression members are objectionable, as they have little resistance against compression. They are easily bent in handling the members, and, even if subsequently straightened, as is a too common practice, they constitute a permanent defect in a bridge.

Special attention was given to uniformity and the neat appearance of the latticing. Main truss members, as well as laterals, have double lattice angles with an inclination of about 45° to the axis of the member. One of each pair of lattice angles is spliced at the intersection by a square plate (Plates XXI and XXII). All latticing is placed inside the flange angles. The tie-plates are as near as practicable to the end of the member, in any case, well within the edge of the gusset-plate, and all cover-plates of the bottom chord are continuous across the panel point, being notched out for the gusset-plates. This is an important improvement over the very common practice of stop-

ping tie-plates or cover-plates outside of the gusset, thus leaving the flange of the member unsupported for a considerable length.

Riveted Connections and Splices of Truss Members.—As yet, little is known about the correct distribution of stresses in a riveted connection. In proportioning, the favorable assumption of uniform distribution of stresses among all rivets of a connection is made, and the secondary stresses in the rivets caused by the stiffness of the connections is generally disregarded, on the assumption that they are fully covered by the margin of safety of the main stresses. In view of the unusually large connections and splices in the main trusses of the Hell Gate Bridge, it was important to guard against local over-stressing.

The following requirements, as quoted from the "Rules of Design," governed the detailing of the connections and splices:

"The strength of connections shall be sufficient to develop the full strength of the member, even though the computed stress is less, the kind of stress to which the member is subjected being considered.

"Truss members with alternating axial stresses caused by live load (including impact) shall be proportioned for the stress requiring the larger sections. The connections and splices shall be proportioned for the larger stress plus 50% of the smaller stress of opposite sign.

"All joints in riveted work, whether in tension or compression, shall be fully spliced, except when differently noted on drawings.

"Rivets carrying calculated stress, and having a grip which exceeds four diameters, shall be increased in number 1% for each additional $\frac{1}{8}$ in. of grip.

"Where splice-plates are not in direct contact with the parts which they connect, the number of rivets on each side of the joint shall be increased over the number theoretically required to the extent of one-third of the number for each intervening plate.

"Rivets carrying strain and passing through fillers shall be increased 100% in number, if the thickness of the filler is equal to the diameter of the rivet, the increase in number being proportionately more or less as the filler shall be more or less thick than the diameter of the rivet."

The further rule was observed to connect, or splice, each individual part of the section of a member as directly as possible, so as to avoid the transfer of stresses through other parts. Typical connections are shown in Figs. 13, 14, 15, and 16.

The connections at each panel point are made with two gusset-plates, 1 in. thick in the middle panels and $1\frac{3}{8}$ in. in the panels nearer the ends. The largest of these are 120 by $1\frac{3}{8}$ in. by 17 ft. 6 in. and

126 by 1⅜ in. by 14 ft. 6 in., and mark the limits of present rolling mill capacities.

The general scheme in detailing the connections of the web members to the gussets was as follows: The web-plate of the member is in the same plane and has the same thickness as the gusset-plate. It is cut off at the edge of the gusset and spliced to the latter by a splice-plate on each side. The flange angles, however, extend as far as possible over the gusset, their outstanding flanges being connected by lug-

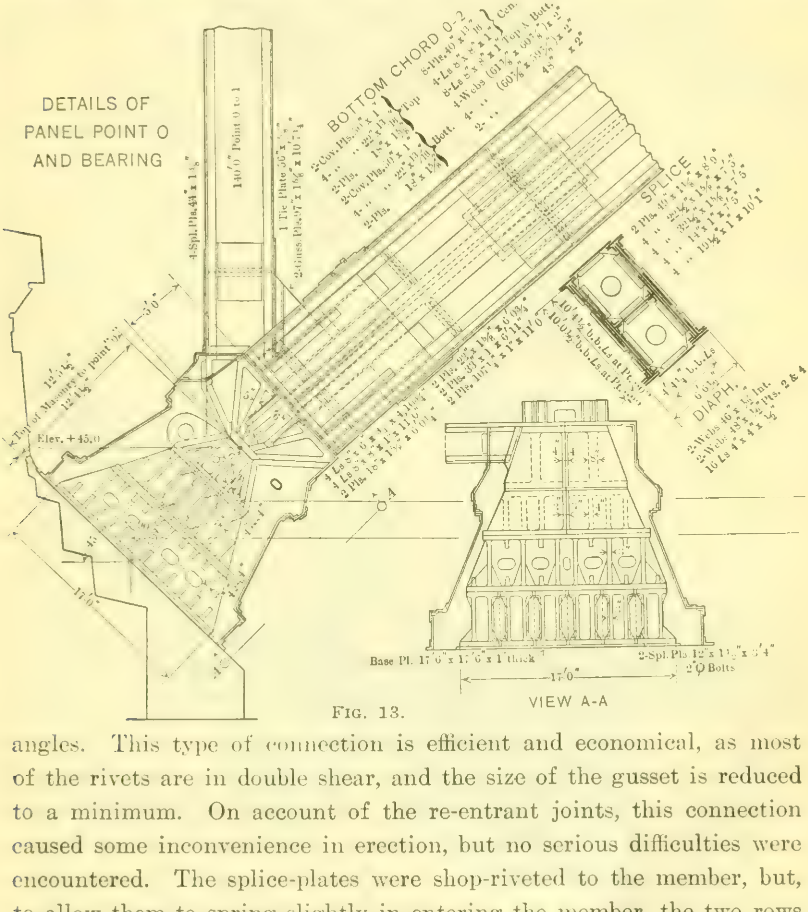


FIG. 13.

angles. This type of connection is efficient and economical, as most of the rivets are in double shear, and the size of the gusset is reduced to a minimum. On account of the re-entrant joints, this connection caused some inconvenience in erection, but no serious difficulties were encountered. The splice-plates were shop-riveted to the member, but, to allow them to spring slightly in entering the member, the two rows of rivets nearest the edge of the gusset-plate were left for field driving.

The top and bottom chords have a butt joint at every panel point on the intersection of the axes of the truss members. The connection

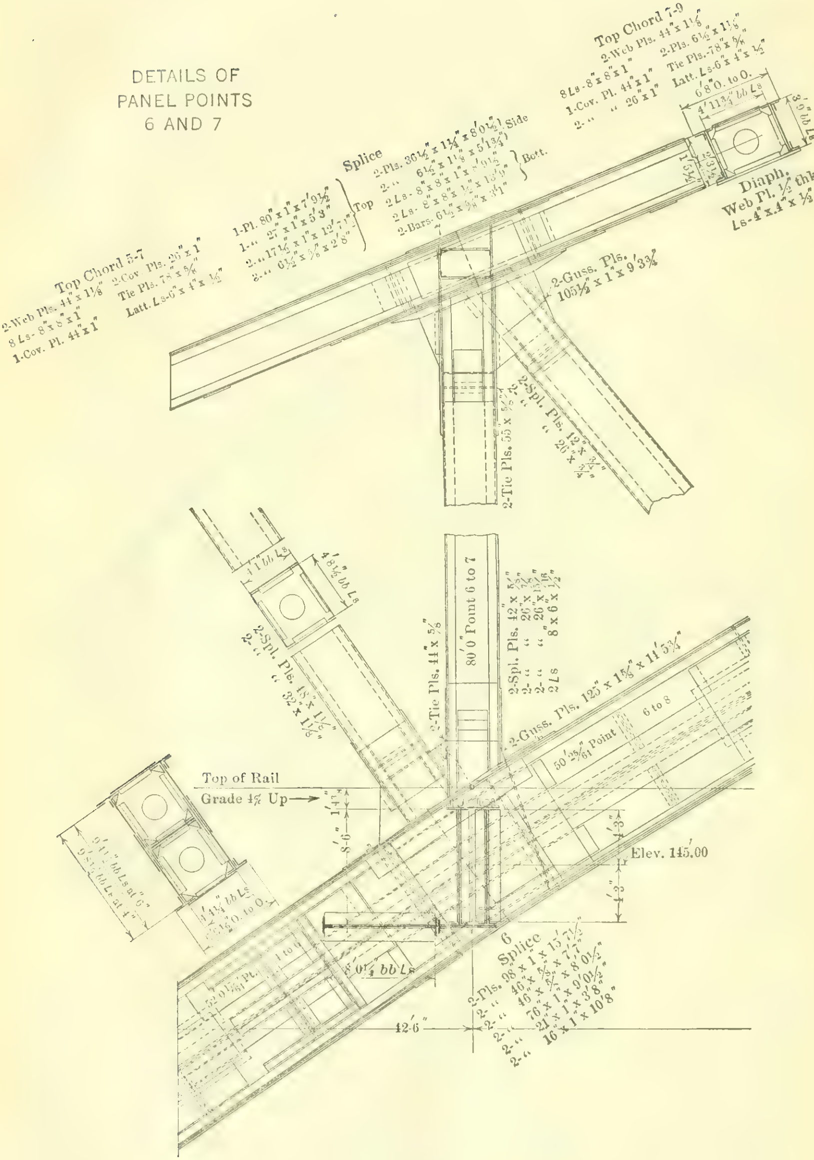

DETAILS OF
PANEL POINTS
6 AND 7
Top Chord 5-7
Top Chord 7-9
Splice
Diaph.
2-Guss. Pls.
Top of Rail
Grade 4% Up
Elev. 145.00
Splice

FIG. 14.

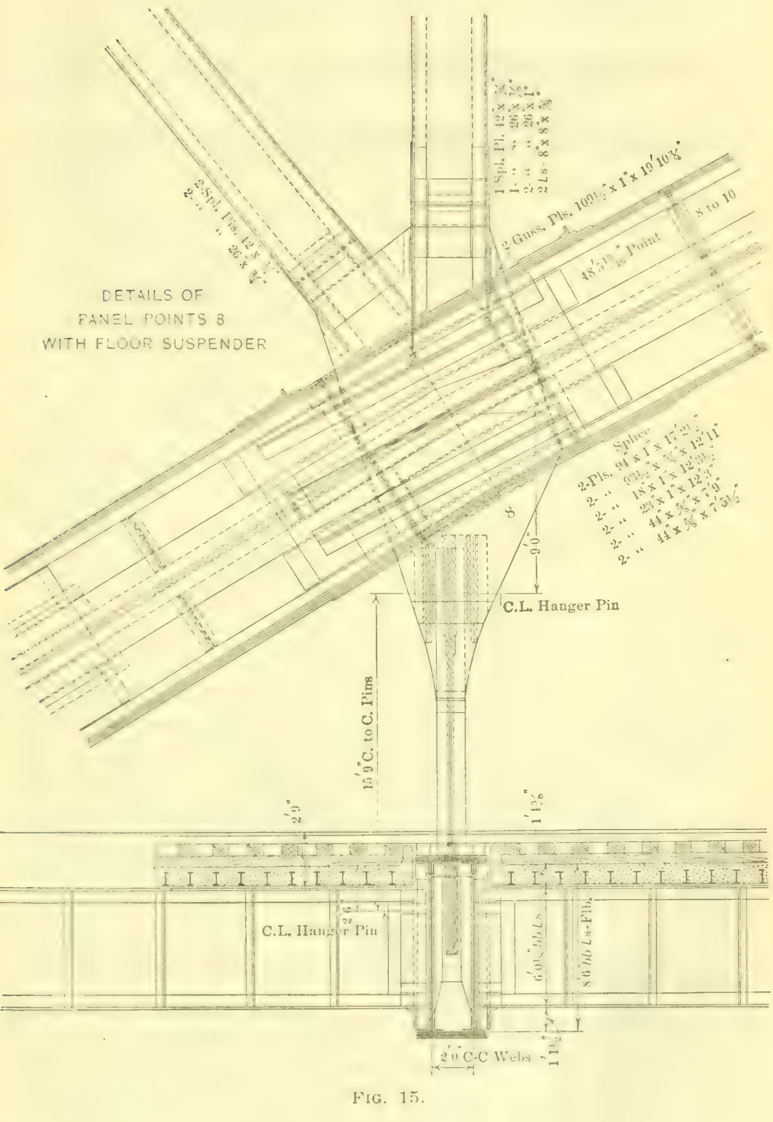

DETAILS OF
PANEL POINTS 8
WITH FLOOR SUSPENDER
C.L. Hanger Pin
15'9" C. to C. Pins
C.L. Hanger Pin
2'0" C-C Webs
Splice
8 to 10
8

FIG. 15.

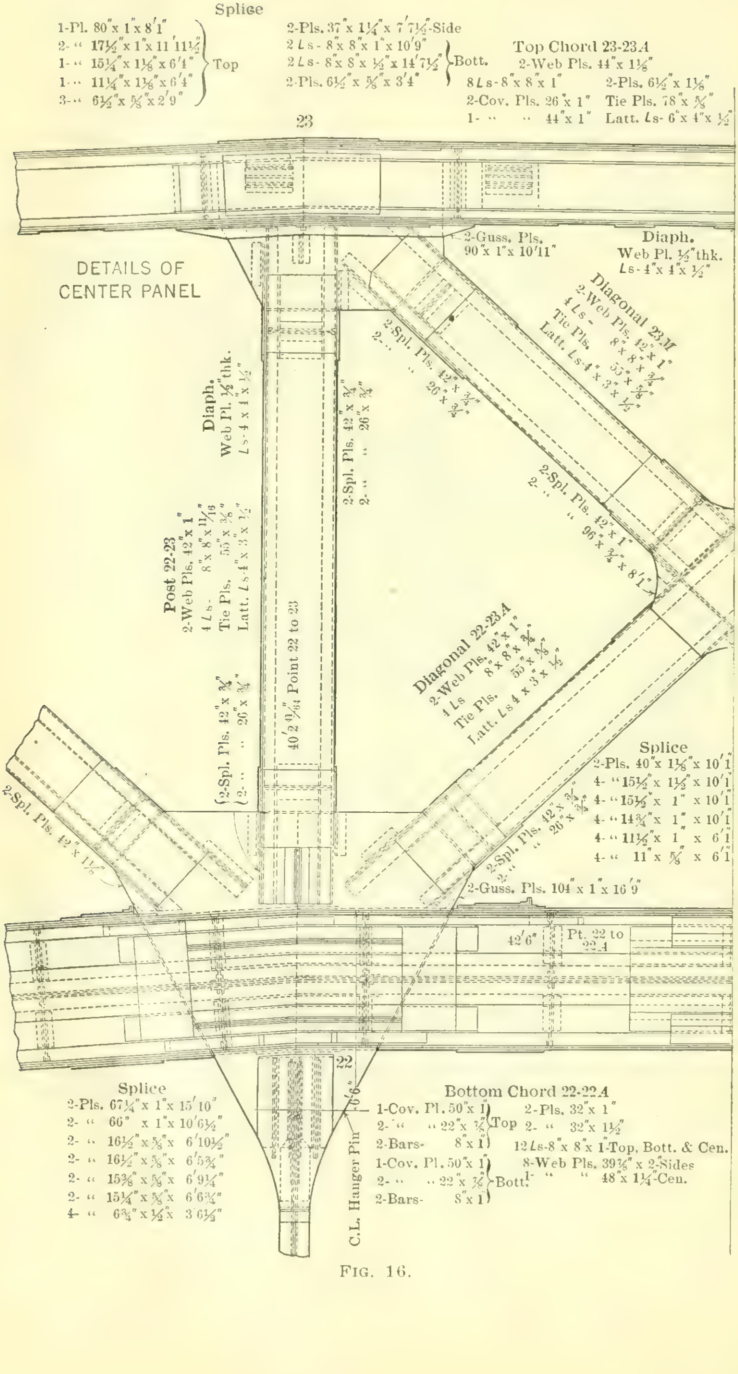
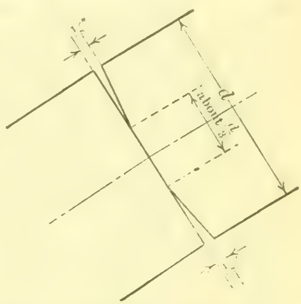

Splice
1-Pl. 80"x 1"x 8'1"
2- " 17½"x 1"x 11'11½"
1- " 15¼"x 1⅛"x 6'4"
1- " 11¼"x 1⅛"x 6'4"
3- " 6½"x ⅝"x 2'9"
Top
2-Pls. 37"x 1¼"x 7'7½"-Side
2 Ls - 8"x 8"x 1"x 10'9"
2 Ls - 8"x 8"x ½"x 14'7½"
2-Pls. 6½"x ⅝"x 3'4"
Bott.
Top Chord 23-23A
2-Web Pls. 44"x 1⅛"
8 Ls - 8"x 8"x 1"
2-Cov. Pls. 26"x 1"
1- " " 44"x 1"
2-Pls. 6½"x 1⅛"
Tie Pls. 78"x ⅝"
Latt. Ls- 6"x 4"x ½"
23
DETAILS OF CENTER PANEL
2-Guss. Pls. 90"x 1"x 10'11"
Diaph. Web Pl. ½"thk. Ls-4"x 4"x ½"
Diagonal 23-M
2-Web Pls. 42"x 1"
4 Ls - 8"x 8"x ¾"
Tie Pls. 55"x ⅝"
Latt. Ls-4"x 3"x ½"
2-Spl. Pls. 42"x ¾"
2- " " 26"x ¾"
Diaph. Web Pl. ½"thk. Ls-4 x 4 x ½"
2-Spl. Pls. 42"x 1"
2- " " 96"x ¾"x 8'1"
M
Post 22-23
2-Web Pls. 42"x 1"
4 Ls - 8"x 8"x 11/16"
Tie Pls. 55"x ⅜"
Latt. Ls-4"x 3"x ½"
40'2 41/64" Point 22 to 23
Diagonal 22-23A
2-Web Pls. 42"x 1"
4 Ls 8"x 8"x ¾"
Tie Pls. 55"x ⅝"
Latt. Ls 4 x 3"x ½"
Splice
2-Pls. 40"x 1⅛"x 10'1"
4- "15½"x 1½"x 10'1"
4- "15⅛"x 1"x 10'1"
4- "14¾"x 1"x 10'1"
4- "11⅛"x 1 x 6'1"
4- " 11 x ⅝" x 6'1"
2-Spl. Pls. 42"x ¾"
2- " " 26"x ¾"
2-Spl. Pls. 42"x 1⅛"
2-Guss. Pls. 104"x 1"x 16'9"
42'6" Pt. 22 to 22A
22
Splice
2-Pls. 67¼"x 1"x 15'10"
2- " 66" x 1"x 10'6½"
2- " 16½"x ⅝"x 6'10½"
2- " 16½"x ⅝"x 6'5¾"
2- " 15⅜"x ⅝"x 6'9¼"
2- " 15¼"x ⅝"x 6'6¾"
4- " 6¾"x ½"x 3'6½"
C.L. Hanger Pin
Bottom Chord 22-22A
1-Cov. Pl. 50"x 1"
2- " " 22"x ⅜"
2-Bars- 8"x 1"
Top
1-Cov. Pl. 50"x 1"
2- " " 22"x ⅜"
2-Bars- 8"x 1"
Bott.
2-Pls. 32"x 1"
2- " 32"x 1½"
12 Ls-8"x 8"x 1"-Top, Bott. & Cen.
8-Web Pls. 39⅞"x 2-Sides
1- " " 48"x 1¼"-Cen.

FIG. 16.

to the gusset is similar to the one described for the web members, except that the web is placed alongside of the gusset, and as all such parts of the section which are not in the plane of the gusset, extend to the joint. Splice-plates are placed on each side of the web and on both sides of the flange material. Thus, again, most rivets are in double shear, and the gusset is utilized as a splice-plate.

The top chords have full bearing at the joint and, in addition, are fully spliced as regards area, section modulus, and value of the connecting rivets. This was necessitated at a few joints on account of tension stresses. However, it also conforms to the best modern practice of splicing compression members fully, and not relying on their bearing at the joint. A splice is not only subject to the direct compression stresses, but also to bending stresses, due to the function of the chord as a continuous column and to the secondary bending moments caused by the rigidity of the connections. Moreover, experience has shown that, unless the chords are very carefully assembled at the shop, no reliance can be placed on getting perfect bearing at the joints.



JOINT OF BOTTOM CHORD

FIG. 17.

Joints in Bottom Chords.—An unusual type of joint was adopted for the bottom chord. The end of one of the chord members meeting at a joint was faced to a perfect plane, and that of the other member was faced to three planes, as shown exaggerated in Fig. 17, so that, when adjoining chord members were assembled, the joint was tight only over the middle third of the depth of the chord, and each outer third formed a wedge-shaped opening ⅛ in. wide. These openings gradually closed during erection as the stress increased and as the assembling bolts and drift-pins were gradually replaced by rivets.

The purpose of this joint was to concentrate the bearing pressure over the middle part of the joint, and thus avoid dangerous edge pressures. Experience with other large bridges fully justified this precaution, especially on account of the unusual width of these chords. The fact, however, that the chords were planed with an accurate, specially built, planing machine, and were assembled carefully at the shop, accounts for the accuracy with which they fitted together in the field.

The outer thirds of the joints are spliced 100%; the middle third is spliced only about 50 per cent. The aggregate area of the splicing material of the whole joint is between 70 and 80% of the effective area of the chord, but its section modulus is nearly equal to that of the main section. Counting in the bearing area of the middle third of the main section, the total resisting area at the joint is from 110 to 120% of the chord section.

Size of Rivets.—In conformity with the great size and thickness of the individual parts making up the members, and to reduce the size of the gusset-plates and splice material to a minimum, it was necessary to use rivets of the largest practicable size. Accordingly, $1\frac{1}{4}$-in. rivets were chosen for the bottom chords throughout, and for the field connections of all other truss members. These rivets have grips up to $9\frac{7}{8}$ in. All other rivets in the top chords, web members, floor-beams, track stringers, and in the heavier laterals are 1 in. in diameter. The rest are $\frac{7}{8}$ in. and less.

Lateral and Sway Bracing Between Trusses.—Each panel of the top and bottom lateral truss has two intersecting rigid diagonals, designed to resist both tension and compression (Plate XXI). There are no transverse struts, except at the panel points near each end, where they form the top and bottom struts of the sway-frames or portals. This lateral system is more rigid and economical than one with struts and slender diagonals designed to resist tension only. All sway-frames have single intersection diagonals. All laterals and members of the sway-frames have a box section, with two or three solid webs, or, in the case of the bottom laterals in the three end panels, one solid middle diaphragm. All open sides have stiff angle latticing.

All solid web-plates are in planes parallel to the plane of the lateral truss or frame to which they belong, so that the laterals can resist effectively the secondary moments and shears due to the distortion of the lateral truss. This principle is well recognized in the design of main trusses, and is equally applicable to the design of bracing in large bridges.

Laterals are commonly proportioned to resist the stresses from wind and other lateral forces. The fact is generally overlooked that the lateral system between compression chords bears to these chords the same relation as the latticing to the different ribs making up a com-

pression member. It forms with the chords a column, which must be strong enough to resist lateral buckling as a whole, and it is evident that inadequate laterals may impair considerably the strength of the bridge as a whole.

A simple approximate rule, similar to that mentioned for the proportioning of the latticing, can be applied to the lateral system, as follows:

> If a is the aggregate gross section of the compression chords of all trusses, in square inches (if the chord section varies, an average value may be taken), the transverse shearing force for which the laterals in any panel and their connections should be designed is, approximately, in pounds, $S = 400\ a$, $330\ a$, and $300\ a$, if the lateral system connects two, three, or four trusses, respectively.

It is not necessary to combine this force with the shear from the assumed wind or other lateral forces, but the laterals and their connections should be strong enough to resist either. This prevents the laterals from being made too light where the wind stress is small.

Arch Bearings.—The four arch bearings are of cast steel (Fig. 13). Each bearing has to transmit to the granite skewbacks a total reaction of 30 262 000 lb., or 700 lb. per sq. in on a bearing area 17 ft. 6 in. square. The upper shoe, which is bolted to the end of the bottom chord, consists of two castings, each weighing 30 tons. The lower face of this shoe is perfectly plane, and bears against the convex cylindrical surface of the lower shoe. This type of bearing produces a rocking motion with little friction under the deformation of the arch.

The radius of the cylindrical surface is $r = 1\,150$ in. The maximum angular motion under live load is approximately $1°\ 30'$ up or down, which produces an eccentricity of only 2.5 in. The pressure per linear inch of line of contact is $p = \dfrac{30\,274\,000}{116} = 261\,000$ lb. Owing to the elasticity of the metal, the contact is actually over a rectangular area, the width of which is approximately $b = \sqrt{\dfrac{p\,r}{E}} = 9.5$ in., wherein E is the modulus of elasticity of the material.

The pressure per square inch increases from zero, at the edge of this area, to a maximum of $s = 0.42\sqrt{\dfrac{p\,E}{r}} = 34\,500$ lb. per sq. in.,

at the center of the bearing. The average pressure is 27 500 lb. per sq. in., which is safe.

The maximum tangential force is 3 570 000 lb., or 12% of the normal pressure, and is easily resisted by the friction. However, to prevent displacement of the upper shoe, four steel dowels, 5½ in. in diameter, are set into the lower shoe and engage holes in the upper shoe.

The lower shoe consists of eleven castings, arranged in three tiers, in which the joints between the individual castings are placed alternately parallel and at right angles to the plane of the truss, so as to insure proper distribution of the pressure. A 1-in. steel plate is placed between the lower shoe and the masonry. Sixteen anchor-bolts, 2½ in. in diameter and 10 ft. long, secure the lower shoe to the masonry.

The total weight of one complete bearing is 248 tons. For better appearance, the whole bearing is enclosed in a steel hood, which produces the effect of a massive pedestal. No mortar or other bed was used between the base plate and the masonry. The skewbacks were carefully dressed to a perfect plane on which dry cement powder was evenly distributed, before the base plate was placed, so as to fill out any unevenness. The holes for the anchor-bolts were drilled to a template after the exact location of the bearings had been fixed.

The failure of the suspended span of the Quebec Bridge, during its erection in 1916, which is ascribed to the breaking of one of the cast-steel bearings, has brought to the foreground the question as to whether cast steel is a suitable material for bridge construction.

Cast steel for bearings has the advantage that it can be built into forms and thicknesses for which riveted work is impracticable. Large steel castings, however, are difficult to make. Experience has shown that unless great care is taken in casting the metal, internal defects may develop, which are difficult to detect. Further, unless properly annealed, as soon as the cores are removed, or after any subsequent local heating, internal stresses will remain in the casting, and may be large enough to cause breakage under slight shock. This applies particularly to complicated castings, or castings in which the metal varies considerably in thickness and, therefore, does not cool uniformly.

Simplicity and uniform thickness, therefore, should be the aim in designing the castings. Provision should also be made to insure quick removal of the cores; otherwise, their resistance to the free shrinkage

of the metal, while it cools, may cause serious stresses in the latter and permanent defects which may not be removed by the subsequent annealing. Bearings which have to distribute a concentrated load over a certain area should have a height of at least one-half the width of the bearing area and be safely proportioned for the bending and shearing stresses. If these precautions are taken in the design and manufacture, and scrupulous inspection is exercised, there need not be any apprehension concerning the use of cast steel for bearings.

Stringers and Floor-Beams.—Typical details of the floor system are shown in Plates XXII and XXIII. The railroad stringers are 6 ft. deep, and of the ordinary make-up. They are framed into the floor-beams. The connection was designed to allow the stringers to be swung into place after the floor-beams were erected. The stringers are connected in pairs by top and bottom lateral bracing and two sway-frames in each panel.

The floor-beams are heavy box girders, with two webs, 8½ ft. deep. The webs are joined by top and bottom cover-plates, 42 in. wide, and by vertical diaphragms in the lines of the stringers and horizontal diaphragms opposite the floor-lateral connections. On the suspended portion of the floor the floor-beams are continuous between railing girders and connected to the floor suspenders by 16-in. pins. At the four end panel points, the floor-beams are framed into the truss verticals, and separate cantilever brackets are provided on the outside. One of the floor-beams weighs 86 tons.

Floor-Lateral Truss and Braking Girders.—The floor-lateral truss is in the plane of the bottom flanges of the railroad stringers, the diagonals being riveted to the stringers at intersection points and also to the floor-beams. The diagonals take tension and compression. Each diagonal consists of a horizontal web-plate with two or four angles riveted to its lower side (Plate XXIII). On account of the great unsupported length between the outer stringer and the wind chord, the diagonals are stiffened in that portion by a latticed strut parallel with and 2 ft. above the diagonal, and connected to the latter by lattice angles.

At the expansion joint of the floor, the two diagonals, on the sliding side, are connected to a horizontal gusset-plate which slides longitudinally, but transmits the lateral reaction to a bracket attached to the center of the floor-beam (Plate XXIII). The chords of the

floor-lateral truss which form the bottom chords of the railing girders have the shape of an inverted T, the vertical web-plate forming the gusset for the web members of the railing girder. To reduce the unsupported length of this chord between floor-beams, the railing girder is connected with the highway stringers by intermediate brackets, one opposite each stringer cross-frame (Plate XXII).

The two braking girders at the intersections of the floor with the trusses are horizontal, single-web plate girders, 8 ft. wide, and placed immediately below the floor laterals (Plate XXIII). These girders are rigidly framed into the bottom chords of the arch trusses (Fig. 14) to which they deliver their reactions. Each railroad stringer is connected to the braking girder by a vertical diaphragm.

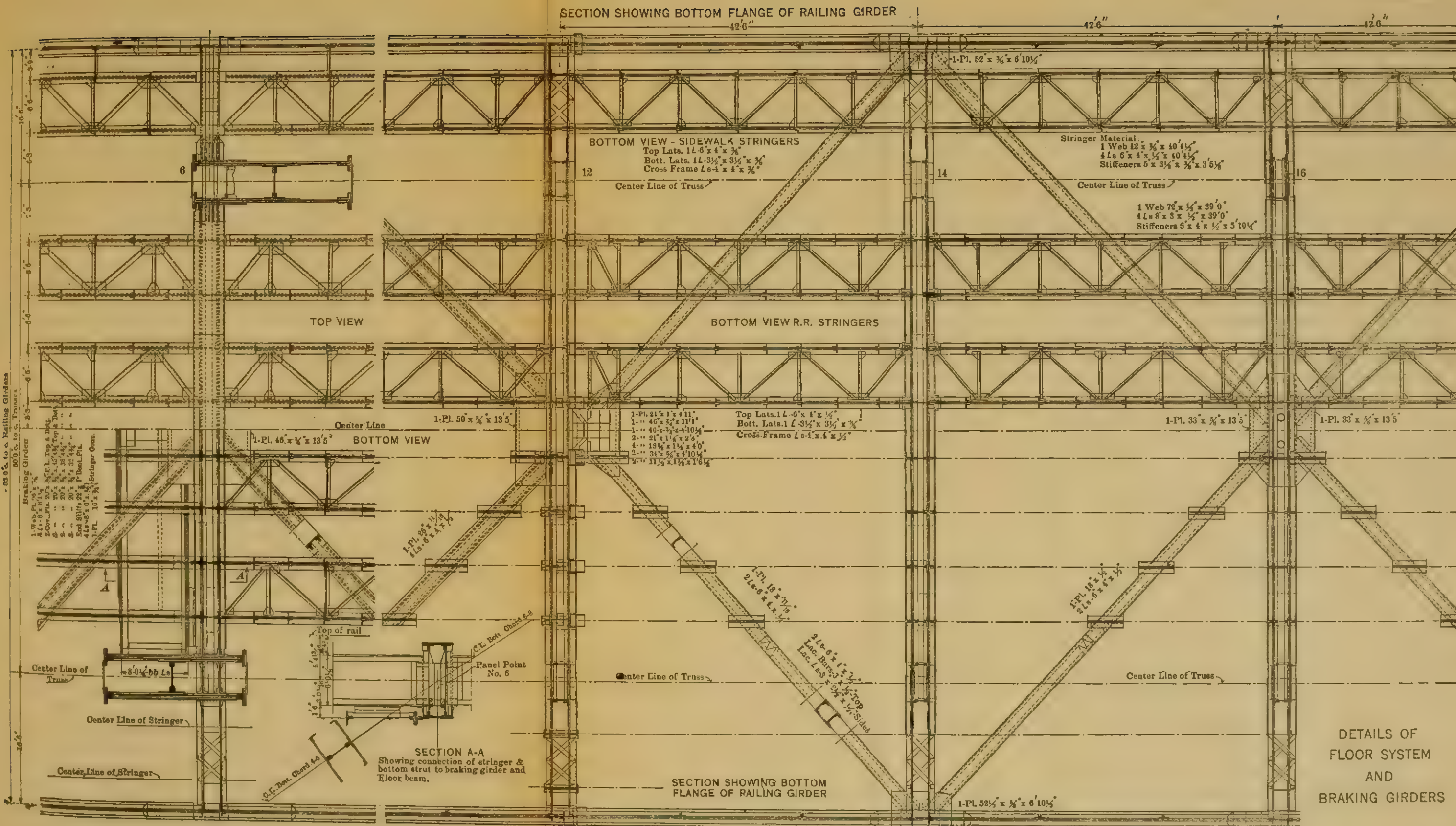
Steel Weights.—The weight of the steel superstructure of the Hell Gate Bridge is made up as follows:

Stringers	4 010 000	lb.
Floor-beams	3 870 000	"
Railing girders with wind chords.....	635 000	"
Floor laterals and braking girders....	575 000	"
Total floor system..........	9 090 000	lb.
Arch trusses......................	21 640 000	"
Floor suspenders..................	1 380 000	"
Arch bracing......................	3 670 000	"
Bearings	1 990 000	"
Total steel weight..........	37 770 000	lb.

or an average of 37 000 lb. per lin. ft. of bridge. This does not include 470 tons of I-beams in the concrete floor-slabs.

7.—Camber and Deformation of Arch Trusses.

For a large bridge of unusual design, an investigation of the elastic behavior of its trusses under load is essential for a proper judgment of its merits. The elastic line gives at once an indication of the comparative rigidity of a bridge, and as to whether and where large secondary stresses may occur. Further, the calculation of the greatest deflection is necessary for determining the proper camber, in order to satisfy clearance conditions under the bridge, and to establish the desired grade of the floor.



DETAILS OF
FLOOR SYSTEM
AND
BRAKING GIRDERS

floor-lat
have th
gusset f
unsuppo
girder i
brackets

The
trusses a
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desire

Camber.—The arch trusses of the Hell Gate Bridge were cambered for dead load only by increasing or decreasing the "geometric length" of each member, including the floor suspenders, by an amount equal—but opposite—to its change in length from its dead-load stress, so that, under full dead load, the trusses would assume their true "geometric form", for which the stresses are calculated. This is the proper method of cambering large bridges. There was no need for cambering the trusses for live load, as the deflections have only a negligible influence on the direct stresses in the truss members. The only object was to prevent the top of rail from sagging below a horizontal line under full live load and extreme low temperature, and this was accomplished by establishing an initial vertical curve for the top of rail.

Deflections.—Plate XXIV shows the assumed initial vertical curve, and gives a summary of the vertical deflections for the points along the top of rail. In the calculation for the deflections, the "gross area" of all members was assumed, and the influence of the details, and of the rigidity of the connections, was neglected. The modulus of elasticity was assumed to be 30 000 000 lb. per sq. in., and the linear temperature expansion as 0.0000065. The effect of the elastic deformation of the web members on the deflections of an arch of this type is very considerable, especially for partial load, and cannot be neglected.

Dead-Load Deflections.—The maximum vertical deflection of the floor, due to the total dead load (average 51 000 lb. per ft. of bridge), from the theoretical cambered position, is 8.35 in. at Panel Point 22 (Wards Island side) and 7.13 in. at Panel Point 22 (Long Island side). This difference for the two sides, and the kink at Panel Point 22, Wards Island side, in the cambered position, is due to the fact that, in its cambered position, the truss forms an unsymmetrical, three-hinged arch, and was erected as such, the hinge being at Panel Point 22, Wards Island side, whereas, in the final position, the arch was to assume its true geometric form, which is symmetrical about the center.

Live-Load Deflections.—Plate XXIV (*b*) shows the elastic lines of the floor for a live load of 12 000 lb. per ft. of truss covering one half span and the whole span, respectively, and also the lines connecting the extreme positions of the panel points under most unfavorable positions of the load. The greatest downward deflection is 5.19 in., or $\frac{1}{2\,300}$ of the span length, and occurs at about the quarter point of the span under a load covering about one-half of the span. The quarter

point on the other half of the span rises 3.46 in. under the same loading condition. The greatest deflection at the center is 3.82 in., or $\frac{1}{3\,100}$ of the span length, and occurs under a load covering approximately the middle half of the span. The maximum live-load deflection at the center of a cantilever bridge of the same span and loading would be about 8 in., and that of a simple span about 5 in.

The elastic lines are smooth curves, or rather polygons, free from local kinks, such as occur at the hinges of cantilevers or three-hinged arches. They are also free from sharp corners, such as are produced over the intermediate supports of cantilevers, or at every panel point of trusses with subdivided panels. From this it may be concluded that the secondary stresses in the arch trusses are comparatively small, which fact is substantiated by the calculations of these stresses, as given in Appendix B.

Temperature Deflections.—A change in temperature from the normal (60° Fahr.) produces an elastic line of approximately parabolic shape with greatest ordinate at the center of the span (Plate XXIV (*c*)). The latter rises or falls, respectively, by 0.74 in. for each degree of rise or fall in temperature, the total deflection for a change of 72° Fahr. being ± 5.3 in., or $\frac{1}{2\,250}$ of the span length. This additional deflection from temperature change is not objectionable, as it has no bearing on the rigidity of the bridge. The temperature deformation is the result of the change in length of each member, first, due to unconstrained expansion or contraction, and, second, due to the elongation or compression from temperature stress. The former effect raises or lowers each point in proportion to its height above the bearings.

8.—Material.

Quality of Steel.—All material in the steel superstructure of the Hell Gate Bridge and Approaches is rolled, forged, or cast steel, made by the open-hearth process.

The following four grades of steel were used:

a.—Hard steel for all rolled parts and pins of the Hell Gate Arch Bridge;

b.—Structural steel for all rolled parts and pins of the Hell Gate Arch Bridge;

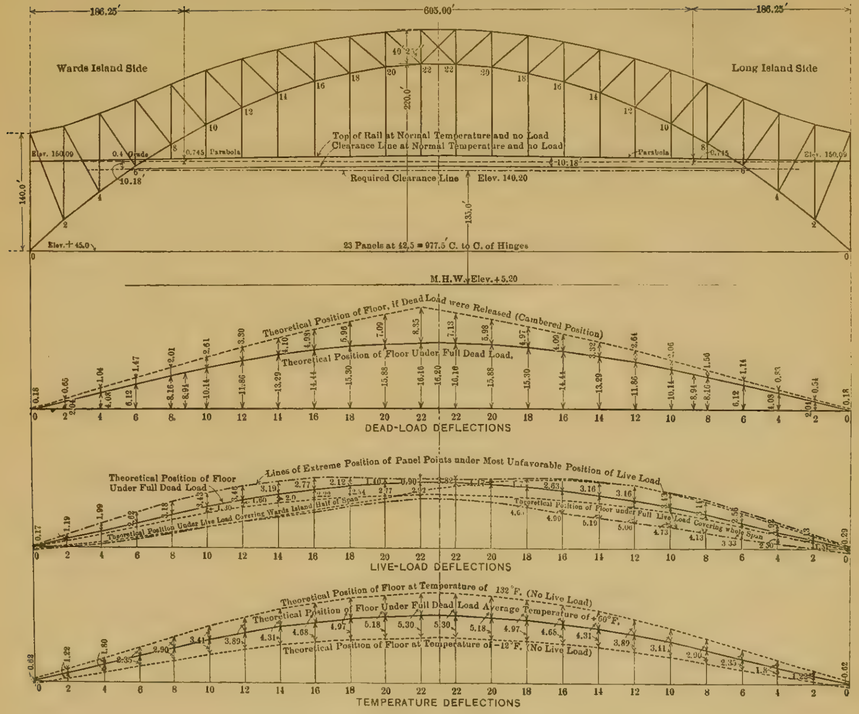
c.—Rivet steel for all rivets;

d.—Cast steel for all castings for bearings, etc.

DEFLECTIONS OF ARCH TRUSSES

Note:- All Deflections in these Diagrams are those of Floor Level, and are given in Inches

+ Denotes Downward Deflections, — Upward Deflections.



These different grades had to conform to the chemical and physical requirements given in Table 1.

TABLE 1.—Chemical and Physical Requirements for Steel.

	Hard steel.	Structural steel.	Rivet steel.	Cast steel.
Phosphorus, max. basic	0.04	0.04	0.04	0.05
Phosphorus, max. acid	0.06	0.06	0.04	0.08
Sulphur, max	0.05	0.05	0.04	0.05
Ultimate tensile strength, in pounds per square inch, max	76 000	70 000	58 000	
Ultimate tensile strength, in pounds per square inch, desired	71 000	66 000		
Ultimate tensile strength, in pounds per square inch, min	66 000	62 000	50 000	65 000
Yield point, min	38 000	35 000	28 000	33 000
Elongation, min. in 2 in. for cast steel; in 8 in. for other steel	*	22%	28%	20%
Character of fracture	Silky.	Silky.	Fine, silky.	Silky or fine granular.
Cold bend without fracture	*	180° around pin of thickness of test piece.	180° flat.	90° around pin of thickness of test piece.

* Minimum elongation for "hard steel": 1 400 000 divided by ultimate strength for thicknesses up to and including ¾ in.; 1% less for each additional ⅛ in. in thickness, with a limit of 16% for thicknesses up to and including 2 in. and 15% for thicknesses greater than 2 in.

Cold-bend test for "hard steel": 180° around a pin of double the thickness of the test piece for material up to and including ¾ in., 180° around a pin three times that thickness for material of greater thickness than ¾ in.

The chemical analysis was made from test ingots during the casting of the melt, and included the determination of carbon, manganese, and silicon, in addition to phosphorus and sulphur. The carbon ranged between 0.27 and 0.34% in the hard steel and between 0.23 and 0.28% in the structural steel, and manganese from 0.52 to 0.64 and 0.36 to 0.61%, respectively. Special stress was laid on sufficient discard being made from the ingot to insure sound material free from piping and excessive segregation.

The physical tests were made on standard test specimens cut from the rolled, forged, or cast piece, in the latter two cases after annealing. The specifications required that at least two tensile and two bending tests be made from each heat of 25 tons or less, at least three tests from each heat up to 40 tons, and four tests for heats exceeding 40 tons. If the material rolled from the same heat varied in thickness ⅜ in., or more, the foregoing number of tests were to be made from the thickest as well as from the thinnest pieces. In all about 7 000 tests, covering

accepted heats only, were made, or an average of eight tests per 100 tons.

Full-Sized Eye-Bar Tests.—About 1 900 tons of structural steel eye-bars, 16 in. wide and from 1⅞ to 2⅜ in. thick, with forged heads 37½ in. in diameter, and 16-in. pin-holes were used in the Little Hell Gate Bridge. Twenty-one full-sized bars were tested, after annealing, and had to show the following results: Ultimate strength: 56 000 to 68 000 lb. per sq. in.; elastic limit: minimum, 33 000, and maximum, 38 000 lb. per sq. in.; minimum elongation in 10 ft.: 12 per cent.

Table 2 gives a summary of the full-sized tests, together with the corresponding unannealed specimen tests.

TABLE 2.—RESULTS OF TWENTY-ONE FULL-SIZED EYE-BAR TESTS.

	Annealed Full-Sized Bars.			Unannealed Specimen.			Difference between average full-size and specimen.
	Min.	Average.	Max.	Min.	Average.	Max.	
Ultimate strength, in pounds per square inch	58 500	61 800	64 700	64 300	67 800	72 600	6 000
Elastic limit, in pounds per square inch	32 900	34 600	36 200	35 700	38 800	41 000	4 200
Elongation: percentage in 10 in	17.8	22.8	29.8	....	....	....	....
Elongation: percentage in 12 in	31.6	39.3	49.8	in 8 in., 21.2	26.4	30.0	....
Reduction: percentage	29.1	36.0	43.2	30.0	36.9	43.0	....

All specimens broke in the body of the bar.

Selection of High-Carbon Steel for Arch Bridge.—A high-grade carbon steel was adopted for the Hell Gate Bridge, principally because it permitted higher unit stresses, which resulted in a considerable saving in steel. The consequent saving in cost was offset only to a small extent by the slightly greater unit price of hard steel over ordinary structural steel. This slightly greater price is not due so much to greater cost of manufacture at the mill and shop as to the fact that, for any special steel, the quantity to be scrapped on account of not fulfilling the requirements, is greater than for ordinary material. The contractor was given the option of furnishing, at the same price, either hard or structural steel for the floor system and suspenders, which parts have been designed with the unit stresses allowed for structural steel. He preferred, however, to use the same material, that is, hard steel, for the whole bridge. Further, with ordinary steel, the sections

of truss members, gusset-plates, splices, and size of rivets would have become much larger and, in some cases, excessive. Even with the hard steel as used the dimensions of some of these details are up to the practicable limits.

The hard steel showed an elongation of generally more than 20%—in the average about 25%—and an average reduction of 45%, and no difficulties were experienced in complying with the bending test. This proves that the material is tough and ductile. It required specially hard tools for drilling and reaming, and these operations were somewhat slower than with ordinary carbon steel, but otherwise no serious difficulties were experienced in the fabrication.

Adaptability of Alloy Steel.—The use of nickel steel for the arch trusses was also taken into consideration, but it was found that, with permissible unit stresses 50% higher than for structural steel, there would have been no saving in cost. The unit price for nickel steel obtaining at that time was about $40 per ton higher than for carbon steel. Moreover, unless nickel steel, or any other alloy steel, would have resulted in an appreciable saving, or other important advantages, carbon steel would still have been given the preference. Carbon-steel trusses, on account of their greater weight or inertia and smaller elastic deformations, are less subject to vibrations from live load, and, consequently, insure greater stiffness and permanency. Moreover, as the proportion of their dead weight to live load is greater, they will sustain safely a comparatively greater future increase in live load than alloy steel.

9.—Loads and Unit Stresses.

The safety and useful life of a bridge depend essentially on the proper assumption and co-ordination of the various loads and permissible unit stresses. In the design of bridges of ordinary span and capacity, the assumption of loads and unit stresses is a matter of well-established routine, whereas, for a large bridge, for which there are few or no precedents, it is a complex problem, which must be solved largely by judgment. The more accurate and complete the load assumptions, the higher can be the permissible unit stresses. It is safer and more in conformity with true economy to make sure that under the most unfavorable, but possible, conditions and combinations of loading the elastic limit of the material or a comparatively high proportion thereof will not be exceeded, than to trust to ordinary

loading conditions and low unit stresses and allow a large, but uncertain, margin of safety. This principle has been followed in the design of the Hell Gate Bridge. All possible forces have been taken into consideration, namely, dead load, live load, vertical impact, lateral force or impact, longitudinal force from traction and braking of trains, wind pressure, and forces due to change in temperature, and comparatively high unit stresses (from five-eighths to three-fourths of the minimum elastic limit) have been allowed for the combination of these forces.

Dead Load.—Before determining on the final sections of the truss members, the dead load was carefully calculated and checked. From the preliminary designs and detailed drawings the panel concentrations were ascertained very closely. After the work had been let a recalculation was made from drawings worked out by the contractor sufficiently in detail to allow the ordering of material therefrom.

The dead load, in pounds per linear foot of bridge, is made up as follows:

Tracks and ballast....................	4 900 lb.
Steel concrete and timber flooring.....	8 100 "
Conduits, cables, wires, etc...........	1 000 "
Total tracks and flooring, etc..................	14 000 lb.
Steelwork, average.........................	37 000 "
Total average dead load per foot of bridge....	51 000 lb.

The dead load assumed for the final calculation is 51 900 lb. average, and varies from 45 000 lb. at the center to 62 000 lb. at the ends. The excess over the actual dead load is due to the excess of the assumed over the actual weight of the conduits. For the weight of steelwork (exclusive of steel in floor slabs), the stresses have been calculated by assuming the arch trusses to be three-hinged, under which condition they were erected (middle hinge at bottom chord, Point 22, Wards Island side, Plate XX). The stresses for the remainder of the dead load, as well as for all other forces, have been calculated for the final two-hinged condition.

Live Load.—The arch bridge and approaches are designed for the following live load:

1.—All bridges and viaducts of the approaches, and the floor system and floor suspenders of the arch bridge, for Cooper's E-60 on each of the four tracks, or an alternative three-axle load of 70 000 lb. on each axle wherever this causes greater stresses (Fig. 18).

2.—The arch trusses for a uniform load of 6 000 lb. per ft. of track, or 24 000 lb. per ft. of bridge, placed in the most unfavorable position in either a single stretch or in two separated stretches, when the latter condition gives a greater stress.

The assumption of a uniform train load for the arch trusses, instead of the engine concentrations, was justified in view of the highly improbable, and almost impracticable, condition of maximum engine and car loads, in the most unfavorable position simultaneously on all four tracks. This hypothetical condition would have required addi-

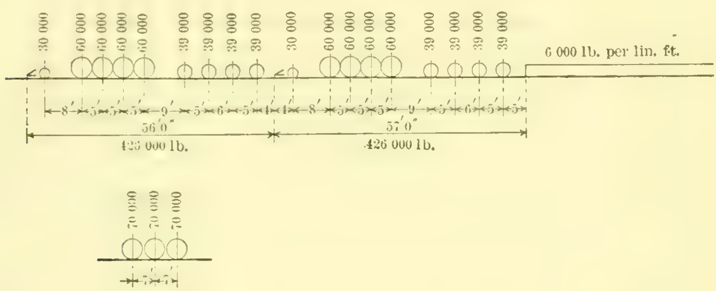


DIAGRAM OF ASSUMED LIVE LOAD

FIG. 18.

tions up to about 5% to the sections of some of the truss members. An E-75 loading on each track in the most unfavorable position, but without impact, would have increased the sections of a few of the heaviest bottom chord members by not more than 3½%, the sections of all the other members being ample. Such a heavy load, if at all possible, with present limits of gauge and clearance, represents probably the heaviest future freight trains, and as these will move comparatively slowly, the impact effect of which on the truss members will be negligible, it is considered, therefore, that the bridge will carry safely any possible future moving load.

Impact.—The impact stresses, or vertical dynamic effects of the locomotives and cars, have been determined according to Lindenthal's formula, as published and fully explained in an article* by its author.

* *Engineering News*, August 1st, 1912.

Since the appearance of that article, the formula has been modified slightly so as to be applicable automatically to bridges having two or more tracks.

The formula in its final form, applied to E-60 loading, is:

$$I = \frac{L^2}{D + L} \times \frac{1\,200 + \frac{a}{n}}{600 + 4a}$$

wherein, D = stress from dead load, in pounds,

L = stress from live load, in pounds.

a = length of train behind locomotive tender for position of maximum stress, in feet,

n = number of tracks loaded for maximum stress.

The modification of the formula consists of the introduction of the value, n, and is based on the assumption of full impact from the locomotives on all tracks, but impact from the cars on one track only. The principal object sought in deriving this formula was that it should be applicable equally to all kinds of bridges, of the shortest as well as the longest spans, to bridges with open or solid ballasted floors, and to steel or concrete bridges; whereas most of the existing formulas have only limited usefulness, particularly the widely adopted formula of the American Railway Engineering Association. The latter formula gives excessive results for long spans and heavy ballasted bridges, and insufficient values for very short stringers, rails, ties, etc., on which the dynamic effect of the driving wheels is unquestionably more than 100 per cent. Lindenthal's formula, in combination with the comparatively high permissible unit stresses, secures economical and well-proportioned structures. Seemingly, it is complicated, but, by tabulation or graphical diagram, its application is very simple.

Lateral Force from Live Load.—The lateral force or lateral impact, due to the swaying motion of fast-moving trains on tangents, or the centrifugal force on curves up to 2°, has been assumed at 600 lb. per lin. ft. of single track. For curves sharper than 2°, 300 lb. were added for each additional degree up to 6 degrees. These forces were increased by 50% for each additional track. This gave, for a four-track bridge on tangent, a total lateral force of 1 500 lb. per ft.

The lateral force occurs simultaneously with the live load, and may act at the same time as the wind pressure, and it is proper, therefore, to provide for it in the proportioning of the laterals as well as the

main truss members. The foregoing value of the lateral force, for a four-track structure, is probably higher than any which may ever actually occur, but it tends to secure a strong and rigid lateral system along the floor. As far as the writer knows, this is the first attempt to make adequate provision for the lateral force from trains on tangents, and to separate it clearly from the wind pressure. The American Railway Engineering Association Specifications of 1914, in this case, would call for a total combined lateral and wind force of only 800 lb. per lin. ft. of floor.

Wind Pressure.—Wind pressure was assumed as a moving load of 500 lb. per lin. ft. of bridge at track level, irrespective of the number of tracks, plus a static load of 30 lb. per sq. ft. on all such vertical surfaces of the unloaded bridge as are exposed at an angle of between 20° above and 20° below the horizontal, or at an angle of 45° from the axis of the bridge. This resulted, for the Hell Gate Arch, in an average wind pressure of 600 lb. per lin. ft. of horizontal projection of top chord, 1 000 lb. per lin. ft. of horizontal projection of bottom chord, and 1 500 lb. per lin. ft. of floor (inclusive of the 500 lb. moving wind pressure), or a total of 3 100 lb. per lin. ft. of bridge. Adding to this the lateral force of 1 500 lb., it is seen that, with a total force of 4 600 lb. per ft. of bridge, ample lateral strength, rigidity, and stability are secured.

Longitudinal Force from Traction and Braking.—The longitudinal force, acting along the rail, from traction and braking has been assumed either at 15 000 lb. for each of the eight driving axles of the two locomotives (25% of the load on each driving axle), or at 1 000 lb. per lin. ft. of train (approximately 15% of the average weight of the train), whichever gave the greater results. For the four-track bridges this force was assumed as acting on two tracks only, in view of the practical impossibility of its acting simultaneously on all four tracks in the same direction.

Recent tests with improved air-brake equipment, yielding a considerably increased brake power, have led to suggestions that structures be designed for a higher braking force than has been the practice heretofore. The effect of the more powerful brakes, that is, the greater force exerted by the brakes to the wheels, is to reduce the time in which a stop can be made without causing dangerous slipping of the wheels on the rails; in other words, the braking force between wheel

and rail attains its maximum value in a shorter time. Its amount, which depends solely on the friction between rail and wheel, is not increased. For reasons of safety, the action of the brakes will never be so sudden that the braking force at the rail acts with impact, that is, the braking force can always be regarded as a static force which increases gradually to its maximum value.

The friction between wheels and rails has been determined variously at from 15 to 30% of the total vertical load on the wheels to which brakes are applied, the percentage depending on the smoothness of the rail surface under varying climatic conditions. As the greatest possible braking force of the two heaviest trains will rarely, if ever, be applied to the bridge, the assumed longitudinal force appears to be ample, no matter how efficient the brake equipment may be made in the future.

Temperature Stresses.—The stresses from temperature have been determined for a variation of ± 72° Fahr. from the normal temperature of 60° Fahr. The temperature stresses are greatest in the center panels of the top chord of the arch trusses, where they attain values of 4 000 lb. per sq. in., or about 40% of the live-load stresses.

Total Stresses.—All members were proportioned for the following combination of stresses:

1.—For members which carry dead and live load, the "total stress" was obtained by adding the stresses from dead load (D), live load (L), impact (I), lateral force (Lat.), and the so-called "excess stress" (Exc.). This excess stress is the sum of the stresses from wind pressure (W), braking force (B), and temperature (T), less 20% of the sum ($D + L + I +$ Lat.). This is equivalent to allowing up to 25% higher unit stresses for the sum ($D + L + I +$ Lat. $+ W + B + T$) than for the previously mentioned "total stress", the percentage decreasing with increasing value of ($W + B + T$).

2.—For members which carry no dead and live load, the "total stress" was made equal to ($W + B + T +$ Lat.). The "total stress," divided by the permissible unit stress, gave the minimum required area.

Permissible Unit Stresses.—The permissible unit stresses assumed are given in Table 3.

The stresses in Table 3 are higher than those allowed by most specifications, notably those of the American Railway Engineering

TABLE 3.—PERMISSIBLE UNIT STRESSES ASSUMED.

<table>
<tr><th></th><th></th><th>For Trusses and Bracing of Hell Gate Bridge. (Hard steel). In pounds per square inch.</th><th>For Approach Spans and Floor System and Suspenders of Hell Gate Bridge. (Structural steel). In pounds per square inch.</th></tr>
<tr><td>Axial tension, net section......................</td><td></td><td>24 000</td><td>20 000</td></tr>
<tr><td>Bending on extreme fiber of beams, girders, and steel castings, net section..............</td><td></td><td>......</td><td>20 000</td></tr>
<tr><td>Axial compression, net section :</td><td></td><td></td><td></td></tr>
<tr><td rowspan="6">(a) Closed section, or section with two diaphragms, or one diaphragm and two planes of latticing........</td><td>$\frac{l}{r}$ = 20</td><td>24 000</td><td>20 000</td></tr>
<tr><td>40</td><td>23 000</td><td>19 000</td></tr>
<tr><td>60</td><td>22 000</td><td>17 000</td></tr>
<tr><td>80</td><td>20 000</td><td>15 000</td></tr>
<tr><td>100</td><td>18 000</td><td>14 000</td></tr>
<tr><td>120</td><td>15 000</td><td>12 000</td></tr>
<tr><td rowspan="6">(b) Half-open section, with one cover and one latticing, or with one diaphragm without latticing.........</td><td>$\frac{l}{r}$ = 20</td><td>23 000</td><td>20 000</td></tr>
<tr><td>40</td><td>22 000</td><td>18 000</td></tr>
<tr><td>60</td><td>20 000</td><td>16 000</td></tr>
<tr><td>80</td><td>18 000</td><td>14 000</td></tr>
<tr><td>100</td><td>16 000</td><td>13 000</td></tr>
<tr><td>120</td><td>14 000</td><td>11 000</td></tr>
<tr><td rowspan="6">(c) Open section, with two or more planes of latticing .</td><td>$\frac{l}{r}$ = 20</td><td>22 000</td><td>19 000</td></tr>
<tr><td>40</td><td>20 000</td><td>17 000</td></tr>
<tr><td>60</td><td>18 000</td><td>15 000</td></tr>
<tr><td>80</td><td>16 000</td><td>13 000</td></tr>
<tr><td>100</td><td>14 000</td><td>12 000</td></tr>
<tr><td>120</td><td>12 000</td><td>10 000</td></tr>
<tr><td>Shearing stress :</td><td></td><td></td><td></td></tr>
<tr><td>On plate girders, net section................</td><td></td><td>......</td><td>15 000</td></tr>
<tr><td>Shop rivets and pins ....</td><td></td><td colspan="2">15 000</td></tr>
<tr><td>Field rivets and turned bolts...............</td><td></td><td colspan="2">12 000</td></tr>
<tr><td>Bearing stress :</td><td></td><td></td><td></td></tr>
<tr><td>Pins.................................. ....</td><td></td><td>24 000</td><td>20 000</td></tr>
<tr><td>Shop rivets.................................. ...</td><td></td><td colspan="2">30 000</td></tr>
<tr><td>Field rivets and turned bolts...............</td><td></td><td colspan="2">24 000</td></tr>
<tr><td>Pressure on :</td><td></td><td></td><td></td></tr>
<tr><td>Expansion rollers per linear inch..........</td><td></td><td colspan="2">Diameter of roller, in inches, × 1 000</td></tr>
<tr><td>Granite masonry........... ....</td><td></td><td>800</td><td>800</td></tr>
<tr><td>Concrete masonry..........................</td><td></td><td>600</td><td>800</td></tr>
</table>

Association. In combination with Lindenthal's impact formula, however, they result in heavier bridges for spans up to about 250 ft. and lighter ones for longer spans. The assumption of the permissible compression stress per unit of net area, that is, with rivet holes deducted, is unusual. Most specifications base the compression stress on the gross section, the assumption being generally made that the rivet shanks replace the compression value of the metal cut out for the holes. This assumption is proper, provided the rivet fills

the hole perfectly, and its metal has the same elastic properties as that of the member. To a certain extent, also, the friction of the rivet heads on the member makes up for loss in compression strength due to the holes.

These conditions, however, are not always fulfilled, and, moreover, compression members are subject to shearing stresses and, near the point of failure, possibly even to tension stresses, both of which cannot be resisted by the rivets, or only to a limited extent. It is evident, therefore, that rivet holes, even after riveting, decrease the strength of a compression member. This decrease may not be appreciable in ordinary compression members in which the rivets have short grips, generally fill the holes well, and develop a comparatively high friction under their head. It is to be assumed, however, that in heavier members the decrease in strength, due to the holes, is greater on account of the longer grip of the rivets and the probability of less perfect rivets.

The extent to which compression members are weakened by the holes can only be determined by a systematic series of comparative tests, embracing both light and heavy sections. In the absence of sufficient experimental data, it is advisable to remain on the safe side.

Another unusual feature of these specifications is the diversity of permissible stresses for different types of compression members, greater stresses being allowed for members having solid diaphragms or cover-plates than for those having flanges which are connected by latticing. The usual compression formulas do not discriminate in this respect, the only provision, in most specifications, is that the portion of the flange between connections of the latticing shall be as strong as the member as a whole. This is usually interpreted to mean that the ratio, $\frac{l}{r}$, of that portion of the flange shall not be less than for the member as a whole, and implies that the strength of the member is the same for any value, $\frac{l}{r}$, of the flange, within the value, $\frac{l}{r}$, of the member as a whole. This assumption is fallacious. The flange of a member, subject to buckling, should have an unreduced compressive value, the same as the flange of a beam subject to bending. If it has not, that is, if the flange itself has a reduced buckling strength, then the strength of the compression member as a whole is doubly reduced.

There are other weaknesses of integral parts, such as insufficient thickness of webs, cover-plates, outstanding flanges of angles, excessive rivet pitch, etc., all of which tend to reduce the strength of a compression member as a whole. It is evidently impossible to cover all these influences by cut-and-dried rules or formulas, particularly in view of the lack of sufficient comparative experimental data, but it is obvious that some distinction should be made in the permissible unit stresses for different types of sections, types of latticing, etc., and not merely for different values of the ratio, $\frac{l}{r}$.

Secondary Stresses.—Care was taken in designing and detailing all the bridges to avoid large secondary stresses and, in a few cases, where this was not possible, additions to the sections were made.

In the Hell Gate Arch the maximum calculated secondary stresses which occur simultaneously with the maximum primary stresses are as follows, in pounds per square inch:

	Dead load.	Live load.	Combined dead and live load.
Bottom chord.......	± 1 300	± 1 300	± 2 100
Top chord..........	+ 2 100	± 3 100	+ 5 200
	— 2 500		— 5 600
Diagonals	± 5 000	± 2 800	± 6 300
Verticals	± 2 200	± 4 400	± 4 800

These, being extreme fiber stresses, can safely be assumed to be covered by the margin of safety of the primary or axial stresses, especially as the greatest secondary stresses occur in members which have considerable excess of section. Moreover, stress measurements made during and after erection indicate that the actual secondary stresses are below the calculated ones.

No attempt, therefore, was made in the fabrication and erection of the trusses to eliminate or even reduce the secondary stresses. In fact, this would have been a very difficult problem on account of the rigidity of the members, particularly of the bottom chord. This question is discussed more fully under "Workmanship and Fabrication."

Erection Stresses.—The greatest stresses in the arch trusses during erection were: + 18 600 and — 16 600 lb. per sq. in. $\left(\frac{l}{r} = 46\right)$ from dead load and erection traveler without wind, and + 20 400 and — 19 700

lb. per sq. in. $\left(\frac{l}{r} = 46\right)$ from the dead load, traveler, and an assumed wind pressure of 30 lb. per sq. ft. of exposed surface.

These stresses were well within the safe limits allowed for the total stresses in the completed bridge. Only in the end panel of the top chord was the erection stress greater than the total stress in the permanent bridge, but these members required a certain minimum section which was sufficient for the erection stress. No extra metal was required, therefore, in the arch trusses for erection purposes. The maximum values allowed for the erection stresses without wind in the temporary back-stays (structural steel) were the same as those allowed for the total stresses in the finished structures (20 000 lb. per sq. in. tension) and 25% more with a wind pressure of 30 lb. per sq. ft. of exposed surface (25 000 lb. per sq. ft tension).

10.—Workmanship and Fabrication.

The specifications were prepared by the Consulting Engineer with a view to securing the best class of workmanship which current practice and possible improvement therein would admit. The steelwork for the arch bridge was manufactured at the Ambridge Plant of the American Bridge Company, whose shop management gave careful study to new shop methods and the use of more efficient tools and machinery, as necessitated for the unprecedented work.

Punching, Reaming, and Drilling.—The punching, reaming, and drilling of rivet holes was governed by the following clauses in the specifications:

"Steel up to and including a thickness of $\frac{1}{2}$ in. may be punched without subsequent reaming, unless the same shall be necessary to insure smooth holes of the assembled parts to be riveted together.

"Steel up to and including $\frac{5}{8}$ in. in thickness shall be punched with holes $\frac{1}{8}$ in. smaller and then reamed after assembling $\frac{1}{16}$ in. larger than the nominal size of the rivets shown on the drawings.

"Structural steel up to and including $\frac{3}{4}$ in. in thickness may be punched and reamed, provided that in no instance shall the punched rivet hole be more than $\frac{11}{16}$ in. in diameter in steel $\frac{3}{4}$ in. thick. This applies to any diameter of rivet, whether $\frac{7}{8}$ in. or larger. After assembling the holes shall be reamed out for the removal of the bruised metal to a diameter $\frac{1}{16}$ in. larger than the nominal size of the rivet. A misfit or overlapping of the punched holes in the assembled pieces greater than $\frac{1}{16}$ in. will not be allowed, and when the Engineer is satisfied, after reasonable trial, that the punching is not within that limit

of accuracy then punching must be discontinued and the holes drilled as per paragraph 55.

"Rivet holes of more than $\frac{5}{8}$ in. in thickness shall be either drilled from the solid $\frac{1}{16}$ in. smaller and then reamed after assembling $\frac{1}{16}$ in. larger than the nominal size of the rivets shown on the drawings, or the rivet holes shall be drilled from the solid after assembling $\frac{1}{16}$ in. larger than the nominal size of the rivet.

"Rivet holes in members composed of material of two or more thicknesses, any one of which is greater than $\frac{5}{8}$ in., shall be punched or drilled, respectively, as specified in paragraphs 54 and 55, and after assembling reamed to proper size, or the rivet holes shall be drilled from the solid after assembling $\frac{1}{16}$ in. larger than the nominal size of the rivet.

"All reaming or drilling of holes in assembled parts shall be done with the various pieces bolted together in their respective relative and exact positions. After reaming, every hole shall be entirely smooth, showing that the reaming tool has everywhere touched the metal.

"All drilling and reaming of holes shall be done dry with self-hardening tool steel.

"After the drilling or reaming is completed, a special reamer shall be run over both edges of every rivet hole to remove the sharp edges and burrs and make a fillet of $\frac{1}{16}$ in. deep in each rivet head. When necessary the work shall be taken apart and any shavings between pieces carefully removed."

On account of the thick material composing the truss members of the arch bridge, nearly all the holes in these members had to be drilled. The general procedure was as follows: After the holes had been laid out with wooden templates and punch marked, from 10 to 15% of all of them were drilled from solid to a diameter of $\frac{11}{16}$ in. The parts were then assembled and bolted up with $\frac{5}{8}$-in. tack-bolts. All the other holes were then drilled from the solid to full size, except those for the field rivets, which were also drilled to a diameter of $\frac{11}{16}$ in. The holes of the $\frac{5}{8}$-in. tack-bolts were then reamed to full size, the bolts being gradually replaced by larger ones, and finally the sharp edges of the holes were reamed off. The member was then riveted.

For the bottom chord members of the arch, the two webs with their flanges had first to be assembled and riveted separately and then joined and riveted to the middle diaphragm and finally the top and bottom cover-plates were added. All reaming and drilling of

the hard steel was done with tools of special high-speed steel. Lubricating with oil was not permitted, but it was found necessary to use occasionally a few drops of soap water to reduce the friction, particularly in drilling through thick material.

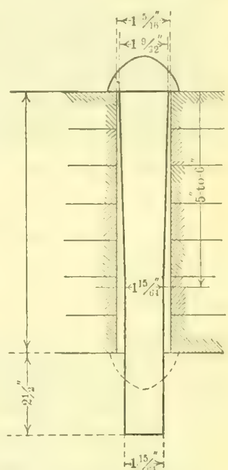
Rivets and Riveting.—There are, in the whole arch bridge, about 840 000 shop rivets and 334 000 field rivets, of which 400 000 are $1\frac{1}{4}$ in. in diameter. Most of the 1- and $1\frac{1}{4}$-in. shop rivets were driven with hydraulic riveting machines having a pressure capacity of 100 tons (100 lb. per sq. in.). The output of one of these machines is about 3 500 rivets per day. For all heavy field rivets and such shop rivets as could not be driven with pressure machines, pneumatic riveting hammers of the No. 90 Boyer type with 9-in. stroke and pneumatic buckers-up were used. The points of the long rivets had to be dipped in water, after heating, so as to insure more complete upsetting of the shank before the head was formed.

The specifications required the rivets to be of such a diameter that it was necessary to force them into the holes with a hammer when hot. Experience has shown that rivets which drop easily into the holes when hot, especially those of long grip exceeding about three times the diameter, cannot be relied on to fill the holes completely after upsetting. Such rivets may seem tight when tested with a hammer, but when cut out often show incomplete upsetting near the middle of the shank and toward the shop-formed head. The specifications further prescribed that rivets with a grip equal to, or exceeding, four times the nominal diameter should be tapered so that the base of the rivet should be $\frac{3}{64}$ in. larger and the point $\frac{1}{64}$ in. smaller than the nominal diameter.

Experiments showed that such rivets, when smooth and of perfect size, and the holes exactly $\frac{1}{16}$ in. larger than the nominal diameter of the rivets or $\frac{1}{64}$ in. larger than the actual diameter at the base, can be driven without scraping the hot metal of the shank and forming a film under the rivet head and fill the holes more perfectly than ordinary cylindrical rivets. It was found, however, that allowance had to be made for the irregularity in diameter of rivets and holes due to the rapid wear of the rivet dies and the drills. After extended experiments, the following size and shape was finally adopted for the $1\frac{1}{4}$-in. rivets (Fig. 19): Minimum diameter under the head, $1\frac{9}{32}$ in., tapered

down to $1\frac{15}{64}$ in. in a distance of from 5 to 6 in. from the head, depending on the length of the rivets, the rest of the shank being cylindrical.

The rivets were required to be perfectly round, and free from loose scale and projecting fins, and before they were entered into the holes, the scale, formed in heating, had to be carefully scraped off. To produce perfectly round and smooth rivets, it was found necessary to deviate from the ordinary practice of upsetting the rivet between dies by a single stroke of the machine and cutting it off at the same time from the feeding rod. Pieces of the proper length required for the various rivets were cut cold from the rivet rods and, after heating, were placed individually into the upsetting machine. Each rivet was given at least two strokes, and was turned after each stroke, so as to make it perfectly round and press down the projecting fins which form in the first stroke at the joint between the dies. Further, by more careful individual handling of the rivets in the heating furnace, it was possible to avoid the scale which often forms on the rivets due to improper heating when handled in bulk. Whatever slight scale was left was removed by running the finished rivets through a rumbling process and, where necessary, projections were removed by grinding.



1¼" TAPERED RIVET
FIG. 19.

Planing Ends of Chord Members.—Great care was taken to secure perfect planing of the ends of the chords. The planing was done by a Bermet Miles horizontal and vertical planer which was especially erected for this work. It is mounted on a rotary platform, and has a cutting range of $12\frac{1}{2}$ ft. horizontally and $10\frac{1}{2}$ ft. vertically.

The three-faced joint of the bottom chord members was obtained by facing the member first to a single plane for the full width. In this condition the chord member was assembled in the truss, and after the truss had again been taken apart the two outer thirds of the face of the chord were planed to the required bevel. About 1 in. of metal was cut away at each face, first by a roughing cut parallel to the vertical webs, and then by a horizontal finishing cut. The length

of the members had to be accurate within $\frac{1}{64}$ in. and the bevels of the faces within 30 sec. (= $\frac{1}{64}$ in. in 10 ft.).

Assembling Arch Trusses.—Complete or partial assembling of riveted trusses, or of continuous chords of pin-connected trusses, is gradually becoming the ordinary method in American practice, at least for important work. With improved facilities for assembling large trusses at the shop, the reaming and drilling of the field connections can be done cheaper than by the formerly prevailing practice of reaming or drilling to iron templates. The assembling at the shop, moreover, insures greater accuracy and decreases the chances for errors or unforeseen difficulties, with the resulting delays and added expenses, in the field.

As the complete assembling of the arch trusses would have required a very large space, and corresponding facilities for handling the members, the Bridge Company was permitted to assemble the truss in sections of four panels, the last panel of each section being again assembled with the following three panels.

Each truss section was laid out carefully in the bridge shop yard to the correct cambered form with a transit. The members were supported by timber grillages of sufficient bearing area to prevent excessive settlements. Levels were taken each morning before any work was done to make sure that the truss section was in a perfect plane during the drilling of the holes. The members were handled with a gantry crane of 130 ft. span and 150 tons lifting capacity (Fig. 20). This crane was erected specially for this work.

As previously mentioned, all holes for the field connections were drilled to a diameter of $\frac{11}{16}$ in. from the solid, and $\frac{5}{8}$-in. bolts were used for assembling. After a truss section was assembled, the holes were drilled to full size with long high-speed drills which reached through both webs. The $\frac{5}{8}$-in. bolts were gradually replaced by larger ones as the drilling progressed, so as to keep the members and gussets always firmly tied together. Before the members were taken apart, the field connections were match-marked carefully and the marks were recorded on a chart for the use of the field force. Fig. 21 shows the special lifting device used for handling the heavy chords.

Only the best class of labor was employed on this work, and the bridge shop deserves full credit for its excellent work, which manifested itself in the accuracy and expediency with which the members

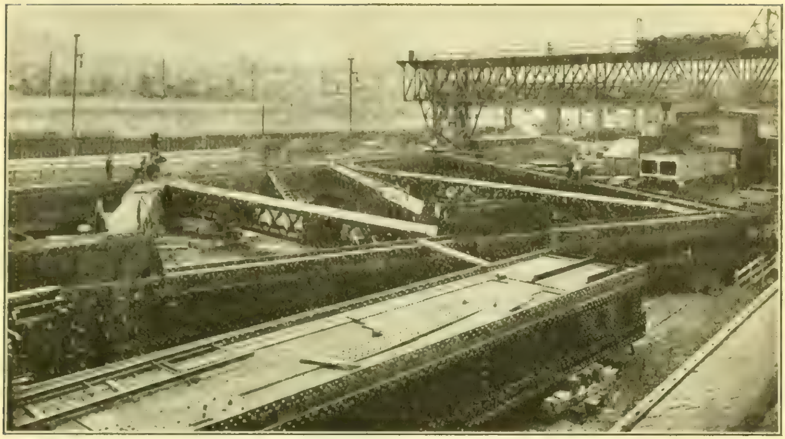


FIG. 20.—ASSEMBLING ARCH TRUSSES AT SHOP, FOR HELL GATE BRIDGE.

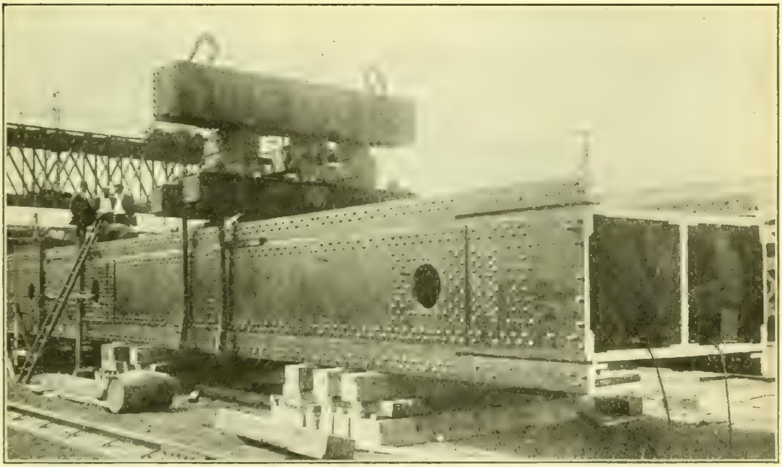


FIG. 21.—BOTTOM CHORD SECTION WITH LIFTING DEVICE, HELL GATE BRIDGE.

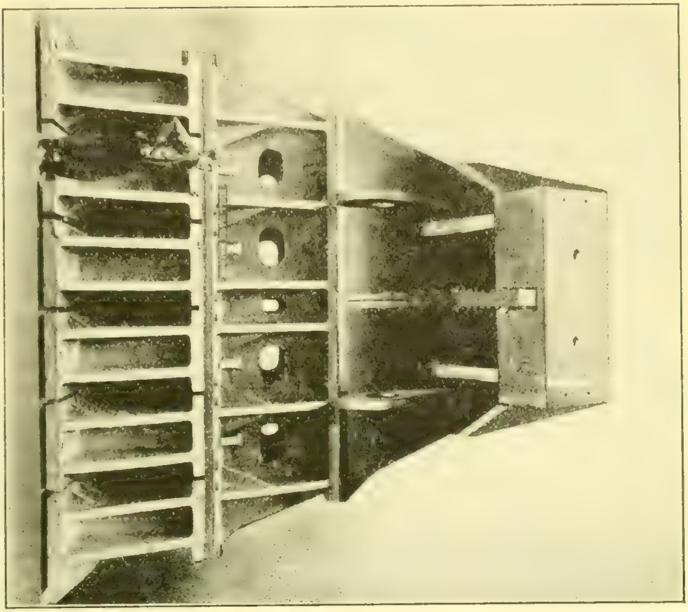


FIG. 22.—ASSEMBLING OF 250-TON ARCH BEARING, HELL GATE BRIDGE.

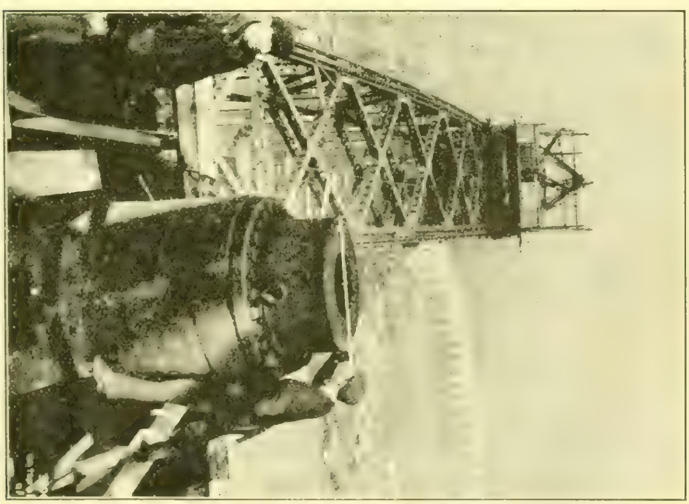


FIG. 23.—3 000-TON HYDRAULIC JACK, HELL GATE BRIDGE.

were erected in the field. The 250-ton cast-steel bearings (Fig. 22) were completely fitted together at the shop.

Angles Between Truss Members and Bevels of Joints.—In the fabrication of large riveted trusses, careful consideration must be given to the question as to whether the angles between members and the bevels of faced ends of chord members shall be made to conform to the "cambered", or to the "geometric", or some other form of the truss. The method to be used depends essentially on the desirability of reducing the secondary stresses, and therefore on the kind and system of truss.

If the angles are laid out to the geometric form of truss, that is, the form which the truss is expected to assume after completion, it is possible to reduce the secondary stresses considerably. This method, therefore, is advisable for trusses with high secondary stresses, such as cantilevers or continuous trusses.

If the angles are made to conform to the cambered form of truss, that is, the form which the truss would assume when entirely relieved of stress, the secondary stresses are, theoretically, fully developed. This method was used for the Hell Gate Bridge, as the secondary stresses are negligible, and because this method has the following important advantages:

First, the whole truss or any number of panels can be completely put together at the shop, with tight joints, and the holes for the connections can be reamed or drilled while the truss is thus assembled, with the greatest possible accuracy and least chance of errors.

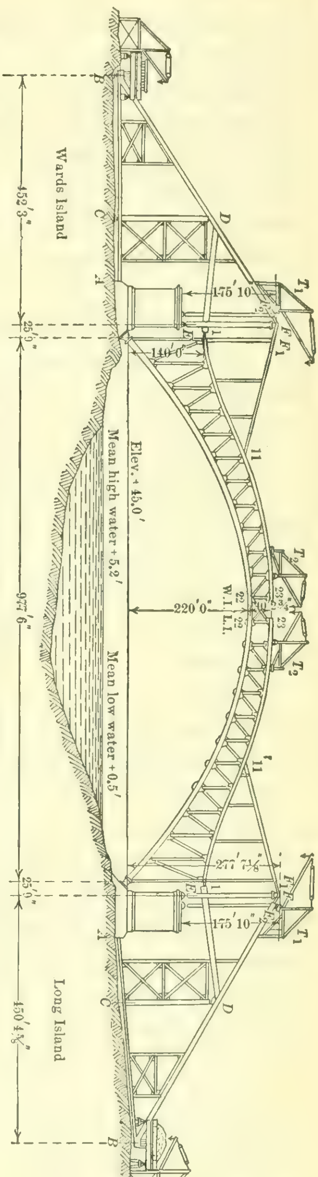
Second, when the truss members are erected in the field, the holes of the connections should match perfectly, and the joints should be tight, without initial bending of the members. The riveting of the connections can start at once, if desired, or the holes can be filled temporarily with tight-fitting drift-pins and bolts to prevent motion of the ends of the member during the deformation of the truss. This secures the greatest safety and expediency in erection.

11.—Erection of Hell Gate Bridge.

Method of Erection.—As is necessary in the case of a large bridge which has few or no precedents, the question of erection was given thorough consideration by the Consulting Engineer, who prepared general erection plans and stress sheets before the design was finally

decided upon. These plans indicated that the erection of the arch was economically feasible, although involving operations of unprecedented character and magnitude. The river conditions, as outlined heretofore, excluded the use of falsework in the river, except for a very short distance from each abutment. Erection on the cantilever principle, with the use of temporary back-stays, was the only practicable alternative. However, as natural anchorages in solid rock for the back-stays were not available at reasonable depth below the surface, the scheme provided for artificial anchorages in the form of huge counterweights to which the back-stays could be attached. To transmit the horizontal pull of the back-stays to the abutment towers, where it could find resistance, struts had to be provided along the ground surface between the counterweights and the towers. To render this method economical, it provided for the temporary use of parts of the permanent bridge in the back-stays.

The general features of this scheme were adopted by the erectors, the American Bridge Company, with whom the responsibility for the erection properly rested. The erectors' plans, which were prepared with a view to secure perfect safety during erection, were approved by the Consulting Engineer after thor-



GENERAL ERECTION DIAGRAM
HELL GATE BRIGDE
FIG. 24.

ough examination and independent calculation of all erection stresses and deflections. The general scheme of erection, as used, is illustrated by Fig. 24.

The Bridge Company resorted to a very skillful utilization of available parts, not only of the arch bridge proper, such as floor stringers, suspenders, etc., but also of the plate-girder approaches, for the construction of the back-stays and counterweights. Only the connections between members, some light bracing, and a number of short eye-bar links were made of extra material, and even of this a considerable tonnage had been used previously on other erection work.

The total weight of steel in the back-stays, for both sides together, amounted to 15 500 tons, of which only about 2 300 tons are not utilized in the permanent structure. The total weight of the arch which had to be supported by the back-stays amounted to 14 500 tons, caused a maximum pull of 6 500 tons in each back-stay, and necessitated a maximum counterweight of 5 300 tons on each side. The heaviest pieces which had to be lifted as units, the bottom chord members of the arch, weighed 180 tons.

Back-Stay Trusses.—The back-stays on the two sides of the river were not alike, on account of different configuration of the surface, and also because the Long Island back-stay had to carry twelve panels of the arch while the Wards Island back-stay carried only eleven. Each back-stay consisted of two separate back-stay trusses placed in the respective planes of the two arch trusses (60 ft. apart on centers). These trusses were designed to resist safely, under the most unfavorable erection conditions, their own weight, that of the arch trusses, travelers, and other erection equipment, and a wind pressure of 30 lb. per sq. ft. of exposed area. Each back-stay truss consisted of the following essential parts (Fig. 24):

1.—The lower back-stay chord, *BD1,* which held the arch truss at the end Panel Point 1 of the top chord during the erection of the first six panels. Except for the short eye-bar links at the connection to the truss, this back-stay chord consisted of two lines of plate girders with an aggregate net section of 316 sq. in. The erection of the arch beyond the sixth panel, with only the lower back-stay chord acting, would have required an excessively large section for this

back-stay and considerable extra material in the top chord of the arch trusses. It was necessary, therefore, to provide:

2.—An upper back-stay chord, *DF*11, which held the arch truss at Panel Point 11 during the erection of the remaining panels, and the closure of the arch. Part, *DF,* of this chord was similar to the lower chord, and part, *F*11, consisted of three lines of floor suspenders. Both parts of this chord were connected to a shoe on top of the column, *EF,* by short eye-bar links, *FF*1 and *FF*2.

3.—The bottom strut, *AB,* which had to transmit the horizontal component of the back-stay chord to the abutment tower, was made up of four lines of floor stringers, braced together laterally, and supported by timber grillages. It had a maximum section of 290 sq. in.

4.—The vertical post, *CD,* had to carry the vertical component from the lower tension chord. The chord was seated on the post by a roller nest in order to prevent serious stresses in the longitudinal bracing between posts.

5.—The vertical post, *EF,* which was seated on top of the masonry tower at track level had to transmit the vertical component from the upper-tension chord. It acted as a rocker, having been provided with pin bearings at top and bottom. This member had the largest section, 348 sq. in., and was made up of four lines of viaduct girders with their bracing.

All other vertical posts, made up mostly of pairs of floor stringers, carried only the weight of the chords and the traveler. All posts were well braced in pairs longitudinally and transversely, and thus formed rigid towers and bents. The bearings of the arch trusses were provided with temporary eye-bar anchorages (Fig. 26) to prevent sliding on the skewbacks during the early stages of erection when the reaction had the steepest inclination.

Counterweights.—The counterweights were carried on top of the rear ends of the back-stay trusses, which rested on a grillage foundation. On the Long Island side the counterweight was made up of three layers of viaduct girders (Fig. 25). The upper layer formed a box which was filled with earth taken from the excavation for the bottom strut. The earth was filled in gradually as the erection proceeded so that there was always ample margin of safety against uplift and at the same time safe pressure on the foundation.

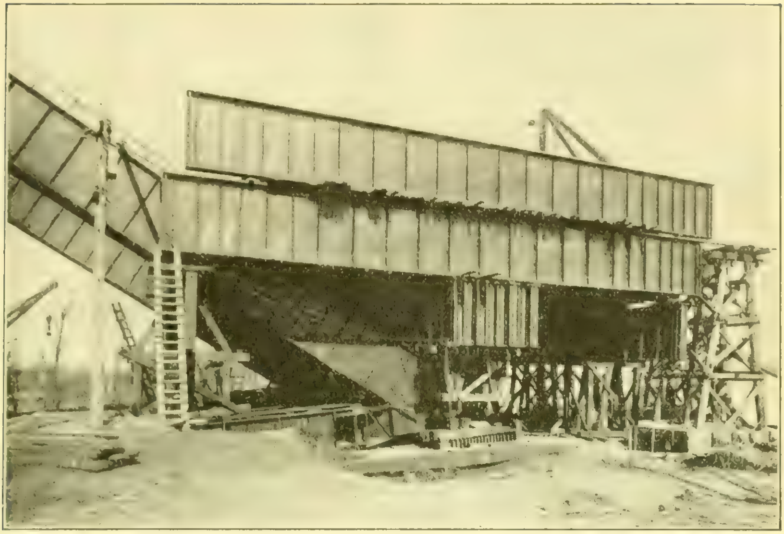


FIG. 25.—LONG ISLAND BACK-STAY COUNTERWEIGHT, FOR HELL GATE BRIDGE.

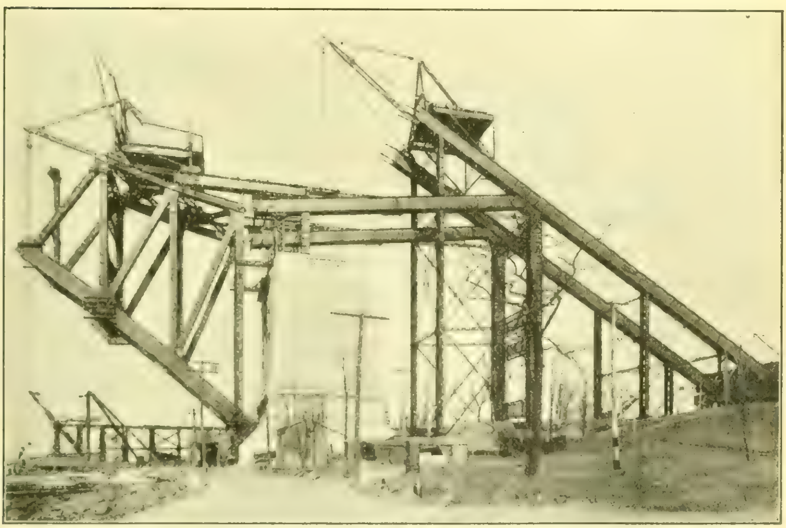


FIG. 26.—HELL GATE BRIDGE: ERECTION STAGE, APRIL 1ST, 1915, LONG ISLAND SIDE.

The counterweight on the Wards Island side was similar, except that, on account of lack of sufficient earth excavation, the necessary weight was made up by additional viaduct girders and flooring I-beams.

To allow free expansion and contraction of the back-stay trusses, and thus prevent large temperature stresses, the ends of the back-stay trusses under the counterweights rested on roller bearings. Provision was also made to raise the counterweights to proper height by eight 500-ton hydraulic jacks (four under each truss) so as to offset settlements of the foundations. These jacks, having been gauged for pressure, served at the same time as a check for the weight of the earth fill.

3 000-Ton Hydraulic Jacks.—Adjustment of the arch trusses in height was required at various erection stages. For this purpose a powerful hydraulic jack (Fig. 23) was placed at the top of each of the four erection posts, *EF* (Fig. 24). Each jack had a lifting capacity of 3 000 tons under a water pressure of about 5 000 lb. per sq. in. The cast-iron jack plungers had a diameter of 39 in. and a maximum stroke of 26 in. The cylinders were of cast steel, and each one was tested at the United States Government testing plant to its full capacity. When operated, the plunger acted against the cast-steel shoe on top of the post, raising or lowering it as desired, and thereby raising or lowering the arch trusses or rather revolving them around their bearings. As the shoe was raised or lowered, shim plates were inserted or removed from between the shoes and their original bearings on the post, so that the jacks could be relieved after the jacking operation was completed.

General Erection Procedure.—The diagrams on Plate XXV illustrate the general procedure and the progress of erection. The erection was to be carried out simultaneously from both sides of the river, but, on account of delays in the construction of the deep foundation of the Wards Island tower, erection on that side was started 4½ months later than on the Long Island side. It was timed, however, so that the two halves of the arch were completed simultaneously. The erection of each half span proceeded in the following order (Fig. 24):

1.—Placing of compression chords of back-stay, on the previously graded surface, with a 60-ton locomotive crane.

2.—Erecting bottom layer of counterweight girders and on top of these the back-stay traveler, T_1, with a 30-ton stiff-leg derrick placed back of the counterweight.

3.—Erecting lower part, $BD1$, of back-stay with the back-stay traveler, T_1, which moved along the top of the back-stay.

4.—Erecting arch traveler, T_2, on top of lower tension chord of back-stay by the traveler, T_1.

5.—Setting of bearings for arch bridge, and erecting first six panels of arch trusses, bracing, and floor-beams, by the traveler, T_2, which moved along the top chord of the arch. At the same time the traveler, T_1, completed the erection of the upper part, DF, of the back-stay, and portion of the tie, F-11, by advancing on top of the back-stay as far as the point, F. The front portion of the tie, F-11, was erected by a derrick mounted on the rear end of the traveler, T_2.

6.—Connecting the upper chord, F-11, to Panel Point 11 of the arch, and raising the point, F, with the hydraulic jacks until the lower chord, D-1, was relieved of its stress and disconnected.

7.—Continuing erection of arch trusses and bracing to the center (eleven panels on Wards Island side and twelve panels on Long Island side) leaving a small gap between the two ends (at bottom chord point 22 WI).

8.—Lowering Points F with the hydraulic jacks until the trusses became self-supporting three-hinged arches.

9.—Moving the travelers, T_2, back to Panel Point 11, where they started the removal of the forward stay and the erection of the floor suspenders and floor-beams, proceeding again toward the center. (The stringers could not be erected in this operation because they formed the compression chords of the back-stays and were at that time not yet dismantled.)

10.—Dismantling of back-stays and back-stay travelers in the reverse order in which they had been erected.

11.—Erecting stringers, railings, and floor bracing of arch by the traveler, T_2, from the center toward the ends. The end panels of the floor were erected, and the travelers, T_2, were dismantled later by a derrick set up at the end of the top chord.

12.—Connecting up of top chord and diagonals in the center panel so as to transform the trusses into two-hinged arches.

FIG. 27.—HELL GATE BRIDGE: ERECTION STAGE, AUGUST 18TH, 1915, WARDS ISLAND SIDE.

FIG. 28.—HELL GATE BRIDGE: ERECTION STAGE, SEPTEMBER 2D, 1915, LONG ISLAND SIDE.

PROGRESS DIAGRAM OF ERECTION OF HELL GATE BRIDGE

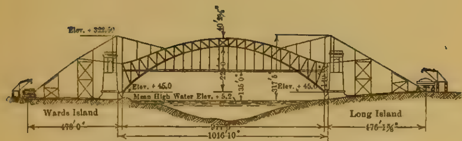
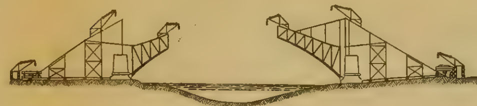


DIAGRAM SHOWING GENERAL DIMENSIONS OF ARCH BRIDGE AND BACK-STAYS

May 31, 1915.

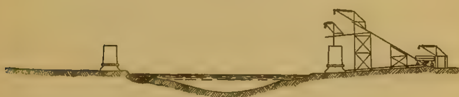
October 31, 1915.

November 30, 1914.



July 31, 1915.

December 31, 1915.



January 31, 1915.

September 30, 1915.

ERECTION OF ARCH BRIDGE AND VIADUCTS COMPLETED OCTOBER 31, 1916.

March 31, 1915.

October 15, 1915.

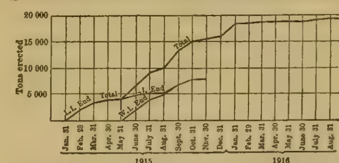


DIAGRAM SHOWING PROGRESS OF ERECTION OF ARCH BRIDGE BY TONNAGE

13.—Placing of concrete flooring, ballast, and tracks, after the riveting of the arch trusses had been completed.

Erection of Truss Members.—The simplicity and uniformity in the design of the various truss panels greatly facilitated and expedited the erection. The typical procedure in erecting a panel of the arch was as follows: The two bottom chords were raised into position, one at a time, the arch traveler standing with its front truck at the end of the previously completed panel. As soon as a chord was in place, the connection at the rear end was made with bolts and drift-pins, and then the falls were released and the chord was allowed to cantilever out. The gusset and splice-plates at the front end of the chord had previously been bolted to the chord on the ground.

Next, the diagonals were raised and connected at both ends, then followed the laterals between bottom chords, the vertical posts, top chord members, and last the sway-bracing between the posts and the top laterals, only one member being raised at a time. On account of the great weight of the chord members it was considered that better progress could be made by raising them individually, instead of in pairs, and therefore the traveler had been designed with a single central boom.

After the completion of a panel, the traveler was moved forward to the next panel point and blocked. The time consumed for the erection of a panel decreased as the erection advanced toward the center, as the erection gangs became more experienced and the members decreased in weight. It took about 3 weeks to erect the end panel on the Long Island side, and the tenth panel on the Wards Island side was completely put up in 7½ hours. The members of the center panel were raised jointly by both arch travelers, the latter standing with their front trucks at Points 21 (Fig. 29). Owing to the accuracy of the shop work, and the careful assembling of the trusses at the shop, no serious difficulties or delays were experienced in making the connections in the field, notwithstanding the unusual number and dimensions of gusset and splice-plates and the fact that no clearance had been allowed for entering parts.

Deflections and Adjustments of Arch Trusses During Erection.—During their erection, the arch trusses passed through the following four principal and distinct static conditions:

1.—Cantilever condition. During erection of first six panels, truss held at end of top chord by lower back-stay (Figs. 26 and 27).

2.—Cantilever condition. During erection of remaining panels, truss held at top chord Point 11 by upper back-stay (Figs. 28 and 29).

3.—Three-hinged arch condition. Back-stays released and trusses connected at bottom chord Point 22 Wards Island side, which acted as hinge, top chord 23-23, and diagonal 23 Wards Island side—22 Long Island side not connected at 23 Wards Island side (Fig. 30). Arch left in this condition until all steelwork had been erected.

4.—Final or two-hinged arch condition. All steelwork erected and all members of the center panel fully connected.

Each transformation from one to the next of these principal static conditions marked a critical erection operation. The first two operations required adjustments of the arch trusses, and to determine the amounts of these adjustments it was necessary to calculate the deflections of certain points of the trusses during erection.

To obtain a clear illustration of the elastic deformation of the trusses during erection, and for the purpose of comparison with observations in the field, complete deflection diagrams for the various erection stages were prepared. The diagrams for the principal stages are shown on Plate XXVI. The deflections of the arch trusses during the cantilever conditions were due, first, to the elastic deformation of the trusses themselves from dead load and weight of traveler, etc.; second, to the elastic deformation of the back-stays from the same loads; and, third, to the change in length of the members from temperature changes. These deflections were very considerable. The total deflection of the bottom chord Point 22, Wards Island side, for instance, under the extreme cantilever condition and at normal temperature (60° Fahr.), was theoretically as follows:

	Wards Island side.	Long Island side.
Vertical downward deflection....	20.6 in.	26.6 in.
Horizontal outward deflection....	7.6 in.	8.1 in.

At a temperature of 30° above normal, these points would have moved 1.6 in. farther out, which would have resulted in an overlap of the two ends of about 19 in. In order that the ends of the two arms should clear each other and that connection at the center could be made by lowering the trusses, it was necessary, therefore, to erect each half in a raised position (revolved around its end bearing).

FIG. 29.—ERECTION STAGE, SEPTEMBER 30TH, 1915. RAISING CENTER PIECE OF BOTTOM CHORD, HELL GATE BRIDGE.

FIG. 30.—HELL GATE BRIDGE: ERECTION STAGE, OCTOBER 4TH, 1915.

Fig. 31.—Hell Gate Bridge: Erection Stage, November 1st, 1915.

Fig. 32.—Hell Gate Bridge: Erection Stage, January 3d, 1916.

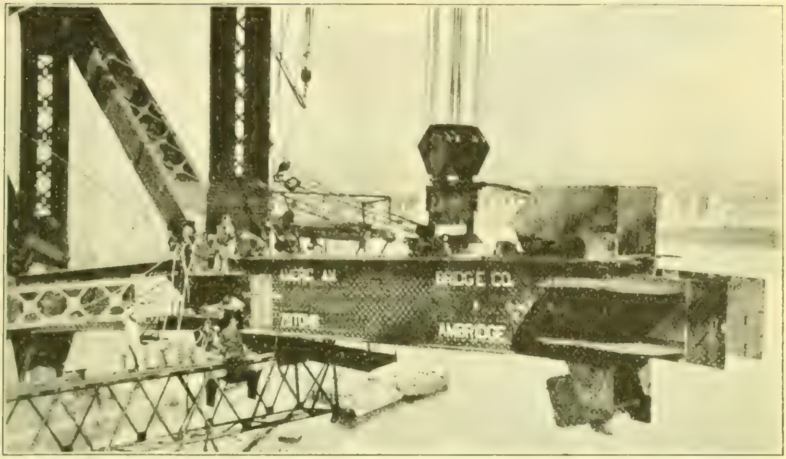


FIG. 33.—CONNECTING BOTTOM CHORD MEMBER, HELL GATE BRIDGE.

FIG. 34.—CONNECTING CENTER PANEL OF BOTTOM CHORD, SEPTEMBER 30TH, 1915, HELL GATE BRIDGE.

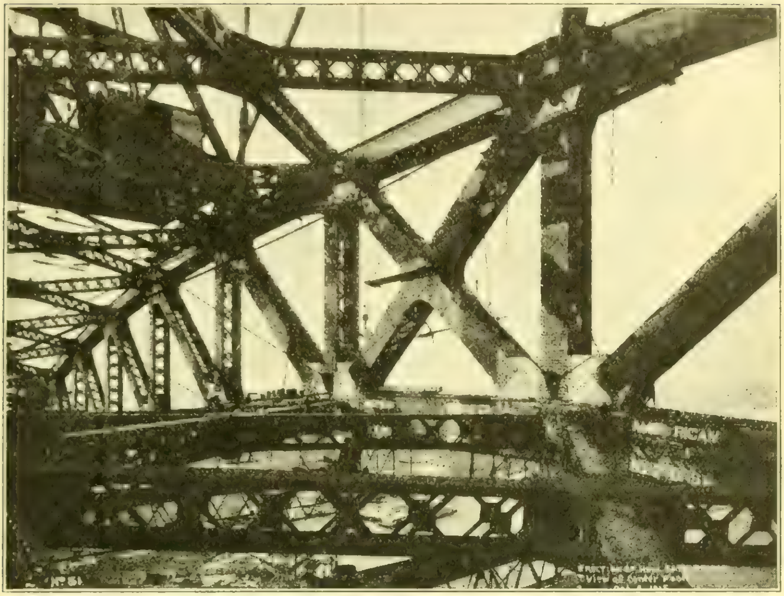
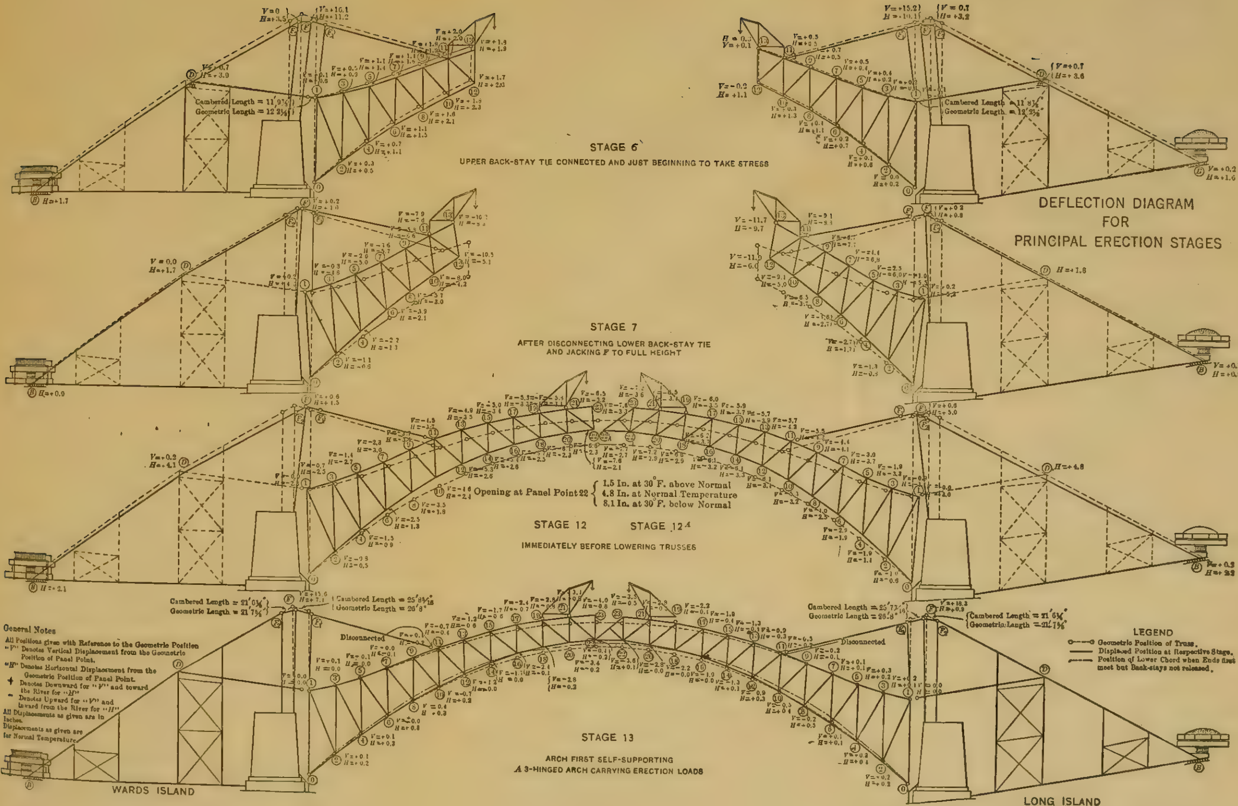


FIG. 35.—HELL GATE BRIDGE: CENTER PANEL COMPLETELY ERECTED, OCTOBER 4TH, 1915.

FIG. 36.—HELL GATE BRIDGE: TRANSPORTATION OF BOTTOM CHORD MEMBERS.


DEFLECTION DIAGRAM
FOR
PRINCIPAL ERECTION STAGES
STAGE 6
UPPER BACK-STAY TIE CONNECTED AND JUST BEGINNING TO TAKE STRESS
STAGE 7
AFTER DISCONNECTING LOWER BACK-STAY TIE
AND JACKING F TO FULL HEIGHT
STAGE 12
STAGE 12A
IMMEDIATELY BEFORE LOWERING TRUSSES
Opening at Panel Point 22
1.5 In. at 30° F. above Normal
4.8 In. at Normal Temperature
8.1 In. at 30° F. below Normal
STAGE 13
ARCH FIRST SELF-SUPPORTING
A 3-HINGED ARCH CARRYING ERECTION LOADS
Cambered Length = 11' 9¾"
Geometric Length = 12' 2¼"
Disconnected
Disconnected
WARDS ISLAND
LONG ISLAND
General Notes
All Positions given with Reference to the Geometric Position
"V" Denotes Vertical Displacement from the Geometric Position of Panel Point.
"H" Denotes Horizontal Displacement from the Geometric Position of Panel Point.
+ Denotes Downward for "V" and toward the River for "H"
− Denotes Upward for "V" and toward from the River for "H"
All Displacements as given are in Inches.
Displacements as given are for Normal Temperature.
LEGEND
Geometric Position of Truss.
Displaced Position at Respective Stage.
Position of Lower Chord when Ends first meet but Back-stays not released.

This position was determined so that, at a temperature of 30° Fahr. above normal, which was considered the extreme high temperature at which the connection might have to be made, there would still be an opening between the ends of the two arms of about $1\frac{1}{2}$ in., to allow for discrepancy between theoretical and actual deflections. That corresponded to an opening of 4.8 in. at normal temperature and 8.1 in. at 30° below normal. The raised position of the trusses was established by making the eye-bar links of the back-stays at Points F shorter than their geometric lengths. Link $FF1$ was shortened by $12\frac{9}{16}$ in. on the Long Island side and by $11\frac{1}{16}$ in. on the Wards Island side, and $FF2$ by $1\frac{3}{8}$ in. on both sides. All other members of the back-stays were made to their true geometric length, and the pin at Point F was assumed to be raised to its normal or geometric elevation. The position for that extreme cantilever condition (Stages 12 and 12*a*) and for normal temperature is shown (by full lines) in the third diagram on Plate XXVI. The positions of the various panel points are given with reference to the geometric position of the arch trusses (shown by dotted lines).

To determine the required range of adjustment by the hydraulic jacks at Points F, that is, the amount by which Points F had to be lowered by the jacks to close the opening at the center and transform the trusses into a self-supporting three-hinged arch, it was necessary to calculate the position of the trusses for the latter condition. That position is shown in the fourth diagram (Stage 13) on Plate XXVI. This diagram shows also the intermediate position (dash and dotted line) which the trusses assumed when the ends of the two arms just came into contact, without transmitting any stress. From the commencement of lowering to this intermediate stage the movement of the trusses was only a downward rotation around their bearings; during the remainder of the movement the trusses changed their elastic form, rising in the center about $1\frac{3}{4}$ in. and sagging at the quarter points about 3 in.

To determine the movements taking place during the operation of releasing the lower back-stay, it was necessary to calculate the positions of the trusses immediately before and after this operation. These positions are shown in the first and second diagrams on Plate XXVI (Stages 6 and 7). The position at Stage 7 is obtained from the position at Stage 6 by raising Point F from an initial low position

to the normal elevation required for Stage 12. To keep the amount of jacking within the range required for the closing operation, it was found necessary to make the eye-bar link, *l-g,* of the lower back-stay shorter than its geometric length by 5¼ in. on the Long Island side and by 4¼ in. on the Wards Island side.

During the first part of the movement from Stage 6 to Stage 7, during which the upper back-stay gradually took its full stress, the trusses changed their elastic form; during the second part, after the lower back-stays had been disconnected, the trusses merely revolved around their bearings. The intermediate stage is not shown in the deflection diagrams. The total vertical movement of Panel Point 11 during this operation was 9.6 in. on the Long Island side and 9.9 in. on the Wards Island side. It is interesting to note, also, the very considerable horizontal deflection of Points *F* at the top of the back-stay column during this operation, amounting to 10.4 in. on the Long Island side and 11.2 in. on the Wards Island side. Fig. 37 shows the theoretical movements of the hydraulic jacks and the positions of the pins, *F,* at the various critical stages.

Jacking Operation for Change from Lower to Upper Back-Stay.—When Erection Stage 6 (Plate XXVI) had been reached, the upper back-stay was connected to the truss at Panel Point 11, with a slight play between the connecting pin and its bearing on the gusset-plates of the truss. The shoes at *F* were then jacked up until the pin at 11 got a firm bearing and the upper back-stay began to take stress (Fig. 37*a*). Jacking was then continued until the upper back-stay had taken full stress and the lower back-stay could be disconnected (Fig. 37*b*). After this the shoes were further raised to their normal elevation (Fig. 37*c*), corresponding to the required position of the trusses for Stage 7. The calculated total raising of the shoes was 22½ in. The shims between the shoe and their original bearings on the post were inserted as fast as the jacking proceeded, and, after completion of the jacking, the jacks were released.

Actually, the procedure on the Long Island side differed somewhat in that the jacking operation was discontinued after the upper tie had taken about 75% of its stress. Erection was then continued until the seventh panel had been placed, after which the jacking operation was completed. The reason for this procedure was that, in changing from Stage 6 to Stage 7, the foundation pressure under the counter-

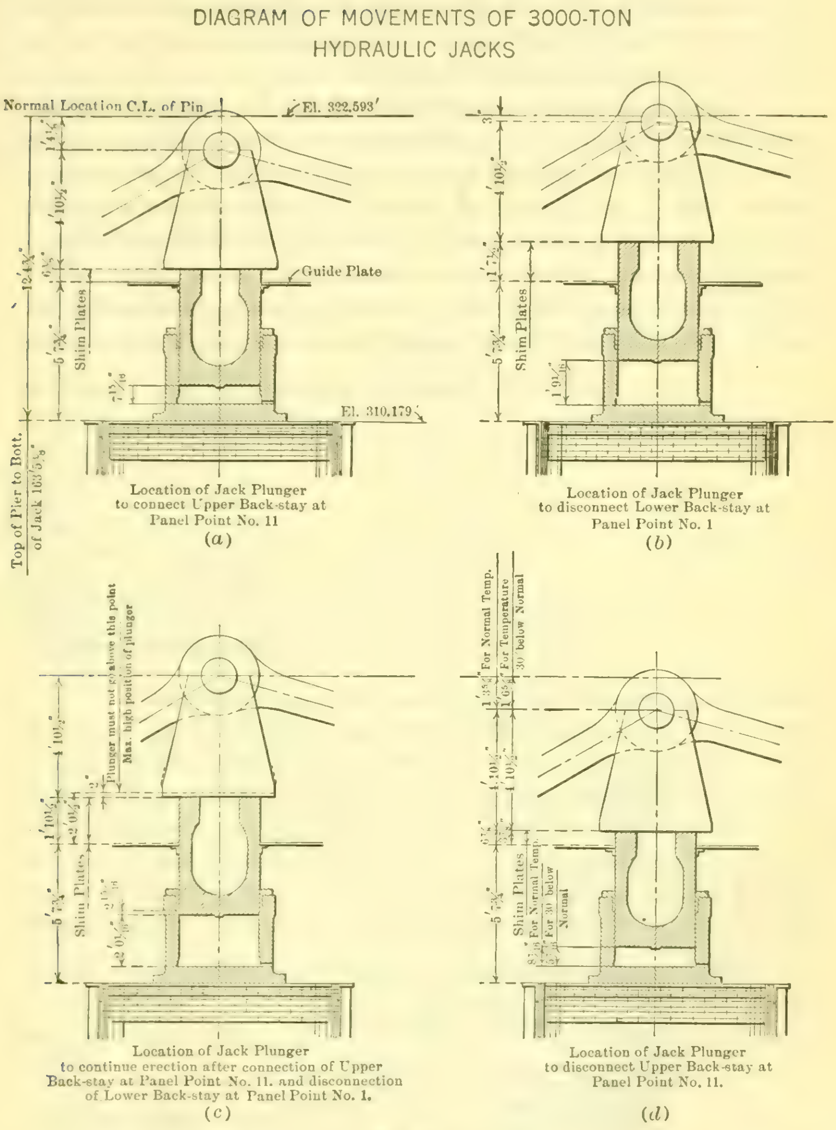


DIAGRAM OF MOVEMENTS OF 3000-TON HYDRAULIC JACKS
Normal Location C.L. of Pin
El. 322.593′
Guide Plate
Shim Plates
El. 310.179′
Top of Pier to Bott. of Jack 163′5⅛″
Location of Jack Plunger to connect Upper Back-stay at Panel Point No. 11
(a)
Location of Jack Plunger to disconnect Lower Back-stay at Panel Point No. 1
(b)
Plunger must not go above this point
Max. high position of plunger
Location of Jack Plunger to continue erection after connection of Upper Back-stay at Panel Point No. 11. and disconnection of Lower Back-stay at Panel Point No. 1.
(c)
For Normal Temp.
For Temperature 30° below Normal
For Normal Temp.
For 30° below Normal
Location of Jack Plunger to disconnect Upper Back-stay at Panel Point No. 11.
(d)

FIG. 37.

weight would have increased from 2.0 to 3.9 tons per sq. ft. This sudden increase might have caused excessive settlement, as the ground was comparatively soft. By adding the seventh panel and moving the traveler to Panel Point 15 before completing the jacking, it was possible to keep the foundation pressure within 3 tons.

A complete record of this jacking operation for both sides is given in Tables 4 and 5. It shows a remarkable coincidence between the jacking heights as calculated and as observed. The "effective" jacking height, that is, the amount of jacking from the moment the upper back-stay started to take stress until the stress in the lower back-stay became zero, was actually 11 in. (north truss) and $11\frac{3}{8}$ in. (south truss), on the Long Island side, and 13 in. (north truss) and 13 in. (south truss), on the Wards Island side, as compared with $10\frac{7}{8}$ in.

TABLE 4.—Record of Jacking for Changing

Date.	Time.	Remarks.	Height of Shims.			Percentage of effective jacking completed.
			Observed:		Calc.	
			N. Truss.	S. Truss.		
			Inches.	Inches.	Inches.	
Aug. 28.	....	Stage 6 —Lower stay fully stressed..........	$3\frac{5}{8}$	$3\frac{5}{8}$	....	....
Aug. 31.	....	Slight jacking to take out slack in upper stay.	$4\frac{1}{4}$†	$3\frac{7}{8}$†	$6\frac{3}{8}$	0
		Continued jacking and stopped at............	6	$5\frac{7}{8}$	....	14
Sept. 2.	8.45	Started jacking at..............................	$5\frac{7}{8}$	$5\frac{9}{16}$	....	13
	9.20	Continued jacking..............................	$14\frac{1}{2}$	$14\frac{1}{8}$	....	77
	9.33	Continued jacking..............................	$16\frac{3}{8}$	$15\frac{3}{4}$	....	90
	9.38	Continued jacking..............................	$17\frac{1}{2}$	$16\frac{5}{8}$	....	97
	9.41	Continued jacking and stopped at...........	$17\frac{5}{8}$	$17\frac{1}{8}$	$19\frac{1}{2}$	100
	9.46	Jacks lowered to bearing on shims...........	$16\frac{3}{8}$	$16\frac{1}{4}$	....	92
Sept. 8.	....	Raised jacks—7 panels erected	17	$16\frac{1}{2}$	....	97
		Continued jacking to remove pins in lower stay	$17\frac{1}{4}$	$17\frac{1}{8}$	$19\frac{1}{2}$	100
		Total effective jacking from 0 to full stress in upper stay	13	$13\frac{1}{4}$	$13\frac{1}{8}$	.....
		Continued jacking after removing pins in lower stay....................................	$22\frac{7}{16}$	$22\frac{3}{8}$	$22\frac{1}{2}$	.....
Sept. 9.	....	Raised jack to level up pins over jacks and at Panel Point No. 11......	$22\frac{5}{8}$	$21\frac{7}{8}$	$22\frac{1}{2}$	.....

* No allowance made for friction. † Interpolated, not observed. ‡ Stress falling rapidly

(Long Island side) and $13\frac{1}{8}$ in. (Wards Island side), respectively, as calculated. The tables also show a close agreement between the tension stresses in the eye-bar links of the upper and lower back-stays as calculated and as actually observed in the field by extensometer measurements. This proves the practicability of such measurements and their value as a means of checking stresses for similar operations. The discrepancy between the calculated jack pressures and the values observed on the pressure gauge is due to the friction between the plunger and the cylinder.

Jacking Operation for Closing Arch.—The closing of the trusses at the center and their transformation from cantilevers into three-hinged arches proceeded as follows: As the Erection Stages 12 (Wards Island side) and 12*A* (Long Island side) (Plate XXVI) had been

BACK-STAYS—HELL GATE ARCH—WARDS ISLAND END.

JACK PRESSURE.		TENSION IN LOWER STAY (EYE-BAR LINKS).			TENSION IN UPPER STAY (EYE-BAR LINKS).			Remarks.
Meas.	Calc.*	Observed: N. Truss.	Observed: S. Truss.	Calc.	Observed: N. Truss.	Observed: S. Truss.	Calc.	
Pounds per square inch.	Pounds per square inch.	Pounds per square inch.	Pounds per square inch.	Pounds per square inch.	Pounds per square inch.	Pounds per square inch.	Pounds per square inch.	
0	0	19 500	19 950	18 160	0	0	0	Before any jacking, 6 panels erected.
....	....	19 500	19 950	18 160	0	0	0	Upper stay begins to act.
500	220	17 100	17 000	16 200	1 050	1 950	1 050	In this condition until Sept. 2d, 1915.
500	200	....	....	....	....	....	....	Traveler moved off jacks before starting.
1 300	1 200	11 400‡	....	4 300	....	....	....	Extensometer readings in lower stay.
1 500	1 400	....	2 700	1 900	....	....	....	Extensometer readings in lower stay.
1 700	1 500	900	....	600	....	....	....	Extensometer readings in lower stay.
1 725	1 550	....	150	0	....	....	7 500	Lower stay eye-bars tested and found slack—STAGE 7.
1 600	1 430	2 400	2 400	1 500	7 650	8 060	8 500	In this condition till Sept. 8th.
1 900	1 660	....	....	600	....	....	8 960	
2 000	1 710	150	....	0	10 200	10 670	9 250	Pins removed to disconnect lower stay.
....	....	....	....	....	....	....	....	
2 050	1 710	....	....	....	....	....	....	
....	....	....	....	....	....	....	....	STAGE 8.—Jacking completed.

reached, the jack plungers were brought to bear against the shoes at Points *F*, which were in the position shown in Fig. 37*c*, and lifted them slightly so that the removal of the shim plates from under the shoes could be started. The jacks were then lowered until the bottom chords came into contact at Point 22, Wards Island side. After lowering a little farther, so as to produce a slight initial stress in the center chord, jacking was stopped. The connection was then made at 22, Wards Island side, with bolts and drift-pins. The jacks were then lowered farther, until the upper back-stay tie was relieved of stress (Fig. 37*d*).

Table 5 gives the record of this operation, and shows, again, a close agreement between the calculated and actual "effective" jacking heights, that is, the amount of jacking from the moment the ends of the two arms came to bear to the moment when the stress in the back-stays

TABLE 5.—Record of Jacking Operation for

Date.	Time.	Stage of closing operation.	Height of Shims.		
			Observed.		Calc.
			N. Truss.	S. Truss.	
			Inches.	Inches.	Inches.
Oct. 1, 1915.	9.30	Position of jacks unchanged since July 23, 1915. Started to raise jacks to loosen shim plates..................................	22 9/16	22 3/8	....
	9.35	Shim plates loose. Started to lower jacks..	24	23 7/8	....
	10.05	Stopped lowering to bolt diagonals at 22 A...	21 1/4	21 1/4	....
	10.23	Started lowering again......................	21 1/4	21 1/4	....
	10.30	Stopped lowering............................	20 3/8	20 3/16	....
	10.53	Raised south jack to match elev. of S. Truss of 22 A..	20 3/8	21	....
	11.00	Started lowering again......................	20 1/8	20 1/4	....
	11.15	Chords touching at 22 A, Temp. 56° Fahr....	19 7/8	19 3/4	18 13/16
	11.28	Stopped lowering to bolt connections at 22 A.	18 3/8	18 1/4	17 3/8
	1.05	Started lowering again and unloading counterweight..	18 3/8	18 1/4	17 3/8
	2.15	Finished lowering jacks, stays 1/4 in. slack. Arch self-supporting, 3-hinged............ Total settlement of counterweight, N = 3 in., S = 1 7/8 in..........................	4 7/8	4 3/4	4 1/16
Summary......		Total lowering of jacks to make chords touch................................	2 11/16	2 5/8	2 5/8‡
		Additional lowering to release stays........	15	15	14 3/4

* Discrepancy between actual and calculated due to trusses being lower than calculated before

became zero. This height was actually 15 in. at both trusses on the Long Island side and $12\frac{1}{8}$ in. (north truss) and $12\frac{5}{16}$ in. (south truss) on the Wards Island side, as compared with $14\frac{3}{4}$ in. (Long Island side) and $12\frac{3}{16}$ in. (Wards Island side), respectively, as calculated. The opening at the center, before lowering started, was actually $4\frac{9}{16}$ in. at a temperature of 56° Fahr., as compared with $4\frac{3}{4}$ in. as calculated for the normal temperature of 60° Fahr. Panel Points 22 were found to be on an average 1 in. lower than the theoretical position, which is a very small deviation for such a great span. A difference in temperature of 10° would have been sufficient to produce that difference in elevation.

Weather conditions were very favorable for the closing operation, the sky having been covered and the temperature nearly constant at 56° Fahr. Great care had to be exercised to bring the ends of the two

CLOSING—HELL GATE ARCH—LONG ISLAND END.

Percentage of effective jacking completed.	Jack Pressure.		Opening Bet. Bottom Chords at Point 22.			Elev. of Panel Point 22 L. I.			Condition of arch.
	Observed.	Calc.†	Observed.		Theoretical.	Observed.		Calc.	
						N. Truss.	S. Truss.		
	Pounds per square inch.	Pounds per square inch.	Inches.		Inches.	Feet.	Feet.	Feet.	
			Actually observed at 56° Fahr.	Corrected to 60° Fahr.		Actually observed at 56° Fahr. / Corrected to 60° Fahr.		Theoretical for 60° Fahr.	
......	....	....	4 9/16	4 1/4	4 3/4*		....		Stage 12 A.
......	4 550	4 600			....				
......	3 950	4 600			....				
......	3 950	4 600			....				
......	3 900	4 600			....				
......	4 500	4 600	1 1/8	1 3/16	1 1/8				
......	3 850	4 600	1 1/4	1 5/16	7/8				
0	3 850	4 600	....	0	0	265.11 265.14	265.14 265.17	.... 265.19	Trusses in contact.
10	3 400	4 100	....	0	0				
10	3 400	4 100	....	0	0				
						265.18 265.21	265.21 265.24	.... 265.30	3-hinged.
100	0	0	....	0	0	Relative Elevations.			
						— 1 1/16 in.	— 3/4 in.	0.0	
......	....	....	....	......	....	....	....	.. ..	
......	....	....	....	....	....	....	....	....	

position. † No correction for friction of jacks. ‡ Calculated for opening observed
jacking.

arms into perfect contact, and, after that, it was important to insure the simultaneous, proportionate lowering of all four jacks, as otherwise serious shearing stresses might have been caused in the center connection. This was secured by a telephone system through which constant communication was kept between the men in charge at the tops of the erection towers, where the jacks were operated, at the center of the arch, and at the main field office.

During the release of the back-stays the foundation pressure under the counterweights would, without reduction of the latter, have been gradually increased from 2.0 to 7.8 tons per sq. ft. This might have caused excessive settlement of the comparatively soft ground on the Long Island side and interfered seriously with the closing operation. To relieve the foundation pressure, the removal of part of the earth fill from the counterweight box was started, therefore, immediately after the trusses had come into contact, and was kept up as fast as possible while the arch was being lowered into the self-supporting position. At the same time, levels were constantly taken to observe the settlements which took place. The average settlement observed was 2½ in., which was not sufficient to cause any disturbance at the center. For emergency the eight 500-ton hydraulic jacks were ready to raise the counterweight, if this had become necessary.

The engineers and field force of the American Bridge Company, particularly C. G. E. Larsson, M. Am. Soc. C. E., Assistant Chief Engineer, who took personal charge, deserve full credit for the well-organized and careful manner in which this critical operation was carried out.

Transforming Trusses from Three-Hinged Into Two-Hinged Arches.—The transformation of the trusses from three-hinged into two-hinged arches required the connection of the top chord and one of the diagonals of the center panel of each truss at normal temperature (60° Fahr.). These members had been erected immediately after the closing of the arch, but had been left bolted at one end and free to move at the other.

The connection at the free end required drilling the rivet holes from the solid, and riveting. Although, during these operations, no elastic deformations of the arch trusses from loads took place, as the members were to be connected without initial stress at normal temperature, it was to be expected that, during the drilling and riveting, which work took several days, there would be changes in temperature.

In order to prevent movements of the connection after the drilling of holes had once started, the Bridge Company resorted to an ingenious device to hold the ends in place. A rod, 8 in. in diameter and 16 ft. long, of sufficient strength to resist any possible tension or compression from changes in temperature, was introduced into each top chord, extending across the open joint of the chord at Panel Point 23, Wards Island side. Each end of the rod was secured with double nuts to a diaphragm riveted to the chord (Fig. 38).

On a favorable day, when the temperature was practically normal and uniform over the whole structure, the nuts on these rods were tightened, with a slight initial stress in the rods. From this moment all stresses were taken by the rods, thus relieving the ends of the chords and permitting the drilling and riveting without disturbance. The rods were left in place, and therefore partake in resisting the chord stresses.

Field Riveting, Bolting, and Drifting.—A total of 333 960 field rivets, or about 17 per ton of steelwork, had to be driven in the Hell Gate Bridge, not including those driven in some connections of the back-stays. About two-thirds of this number, or 202 404, are $1\frac{1}{4}$ in. in diameter and have grips up to $9\frac{3}{8}$ in. Their shape has been described under "Workmanship and Fabrication." The connections of the back-stay trusses were in general made with 80% of drift-pins and 20% of bolts.

All truss connections were made temporarily with bolts and drift-pins. From 25 to 60% of the rivet holes were filled with drift-pins or a number sufficient to carry the entire erection stress. About 50% of the holes were filled with bolts to tie the different parts firmly together. The bolts were not supposed to transmit any stress. The riveting of the connection of the web members and of the six end panels of the top chord was permitted to proceed by gradually replacing the pins and bolts with rivets after the erection had proceeded at least three panels ahead. The riveting of the bottom chord connections and of the middle panels of the top chord, however, was deferred until the arch had become self-supporting and the bottom chord carried the greater part of the dead load. The object was to allow the joints of these compression chords to come into full and forcible contact, and transmit a greater unit stress than the splicing material after the riveting had been completed. As the bottom chords had been planed

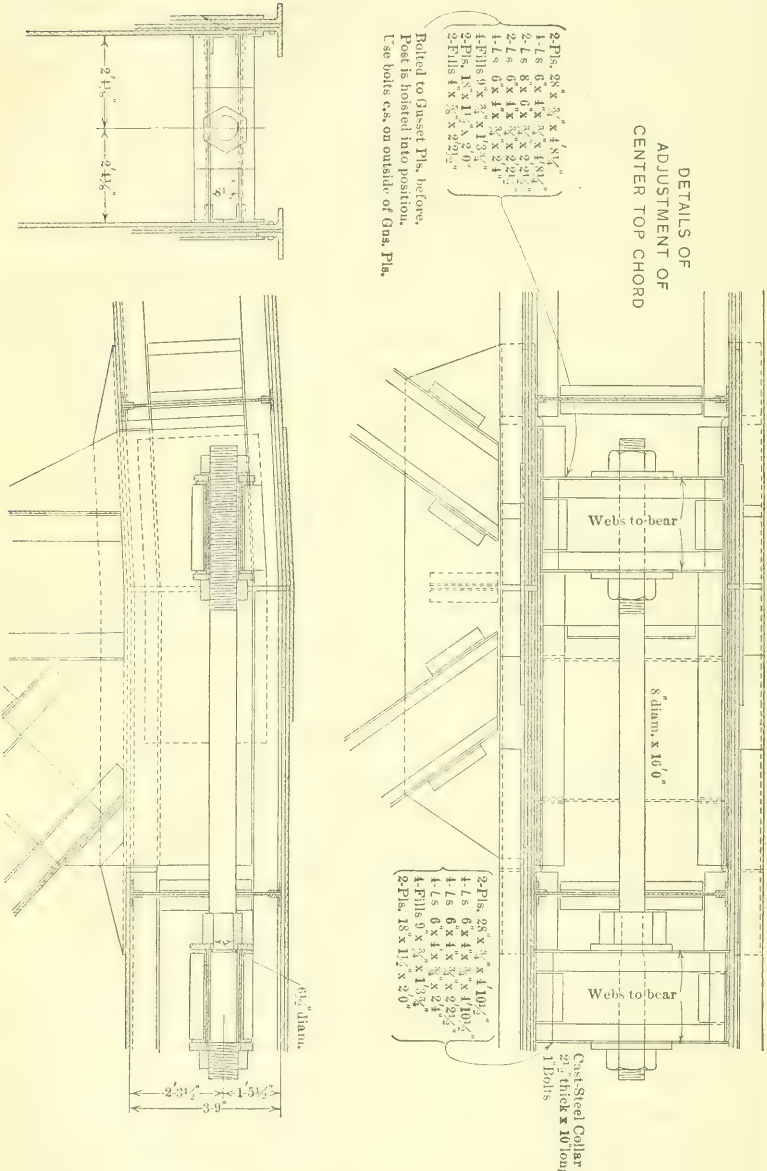

DETAILS OF
ADJUSTMENT OF
CENTER TOP CHORD
2-Pls. 28" x 3/4" x 4'8 1/4"
4-∠s 6" x 4" x 3/4" x 4'8 1/4"
2-∠s 8" x 6" x 3/4" x 2'2 1/2"
2-∠s 6" x 4" x 3/4" x 2'2 1/2"
4-∠s 6" x 4" x 3/4" x 2'4"
4-Fills 9" x 3/4" x 1'3 3/4"
2-Pls. 18" x 1 1/2" x 2'0"
2-Fills 4" x 5/8" x 2'2 1/2"
Bolted to Gusset Pls. before
Post is hoisted into position.
Use bolts c.s. on outside of Gus. Pls.
Webs to bear
8" diam. x 16'0"
2-Pls. 28" x 3/4" x 4'10 1/2"
4-∠s 6" x 4" x 3/4" x 4'10 1/2"
4-∠s 6" x 4" x 3/4" x 2'2 1/2"
4-∠s 6" x 4" x 3/4" x 2'4"
4-Fills 9" x 3/4" x 1'3 3/4"
2-Pls. 18" x 1 1/2" x 2'0"
Webs to bear
Cast-Steel Collar
2 1/2" thick x 10" long
1" Bolts
2'1 1/8"
2'1 1/8"
6 1/2" diam.
2'3 1/2"
1'5 1/2"
3'9"

FIG. 38.

at one end to a three-plane face, as explained previously, the two outer thirds of each joint formed wedge-shaped openings when the member was put in place. As had been anticipated, the openings gradually closed as the stress increased, and particularly when the drift-pins were replaced by rivets. It may be assumed, therefore, that the bearing stress across the joint from dead load increases from nearly zero at the edges to a maximum over the middle third of the joint, and that dangerous edge pressures are thus avoided. Part of the dead-load stress, of course, remained in the splice material, as the gradual replacing of the drift-pins by rivets did not entirely relieve the splice material from stress. The additional stresses from live load and other forces are shared proportionally by the full joint and the splice material.

All field riveting was done by pneumatic hammers with pneumatic buckers-up, as described elsewhere. Air was delivered, with a pressure at the tool of about 120 lb. per sq. in., from two compressors, one on each side of the river. From twelve to seventeen riveting gangs were employed on the bridge. The average daily output for rivets 1¼ in. in diameter was 135 per gang and the maximum daily output of one gang was 356.

Storing, Shipping, Unloading, and Handling Steelwork.—Material from the shops was delivered by rail, and stored at the Pennsylvania Railroad freight yards at Greenville, N. J. Special low cars were required for the transportation of the heavy, deep chord members (Fig. 36). From Greenville the material was re-shipped on car-floats up the East River to the bridge site as needed during the erection.

On both shores of the river a dock had been built for unloading the materials, each having been provided with a 65-ton double, stiff-leg derrick. These docks were also used for delivering materials for the masonry. The members for the three end panels of the arch were raised from the ground or dock by the arch traveler (Fig. 26), those for the other panels were floated on a barge to a position under the arch traveler, and raised by the latter directly from the barge (Fig. 28). To facilitate and expedite the lifting and putting into place, a special hitch was temporarily attached to each heavy member above its center of gravity. This was placed so that the member would hang in the same relative position as it was to occupy in the structure.

Travelers.—The four powerful creeper travelers, one pair on the back-stays and one pair on the arch proper, had been specially designed

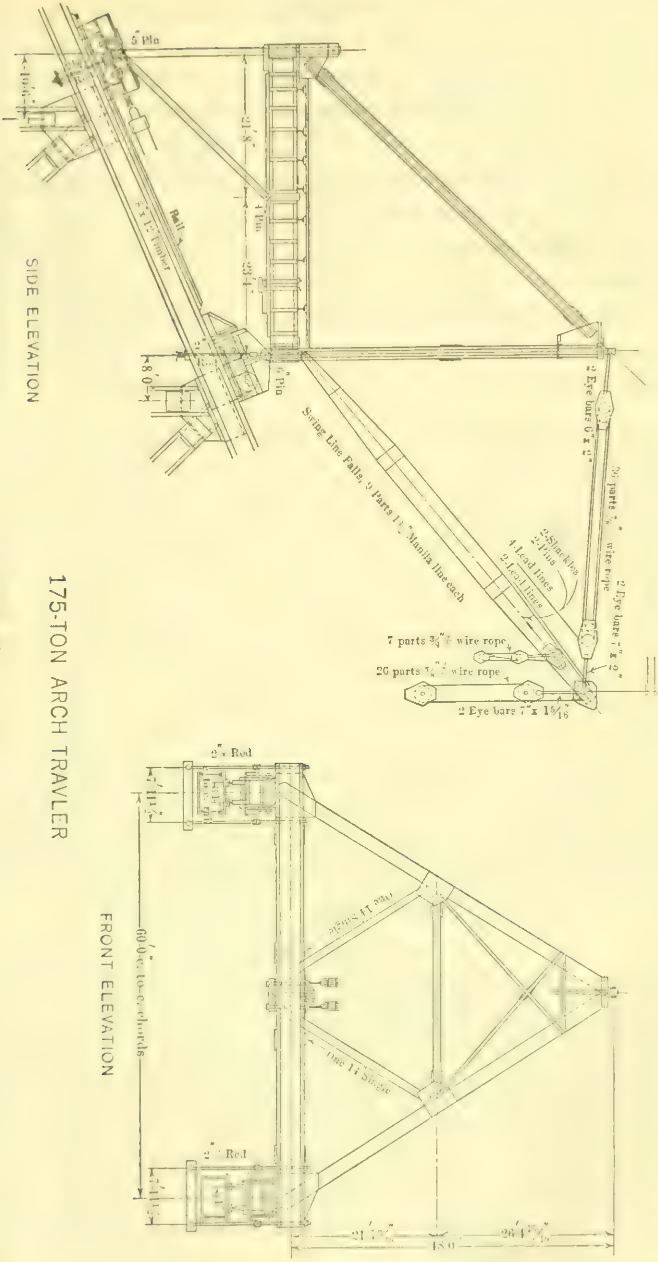
and built for the erection of the arch bridge. They were of the pyramidal shape, with a vertical **A**-frame over the front trucks and a single central boom, except that the arch travelers were also provided each with two light auxiliary booms which carried the working platforms, or cages, and other light loads (Fig. 26). Fig. 39 shows the details of the arch traveler.

On account of the variable inclination of the chords over which the travelers had to run, the rear corners of the traveler platform rested on telescoping columns which were adjustable in height so that the platform could always be kept horizontal. The following are the principal data:

	Arch traveler.	Back-stay traveler.
Maximum lifting capacity.	175 tons (54 ft. radius) (45 ft. side reach).	62 tons (64 ft. radius) (50 ft. side reach)
Heaviest pieces lifted.....	180 tons.............	62 tons.
Total weight, inclusive of equipment	315 tons.............	166 tons.
Height of **A**-frame.......	48 ft................	53 ft.
Length of boom.........	65 ft................	112 ft.
Main falls...............	26-part $\frac{7}{8}$-in. wire rope.	12-part $\frac{3}{4}$-in. wire rope.
Boom falls..............	36-part " " " .	26-part " " "
Falls for moving traveler...	Two 12-part 2-in. manila rope....	Four 12-part 2-in. manila rope.
Maximum lift............	300 ft................	300 ft.
Motors..................	Two 240-h.p. electric...	One 100-h.p. electric.

The back-stay travelers, with some modifications, were used subsequently for the erection of the Wards and Long Island plate-girder viaducts.

Power Plant.—Electric power was used exclusively for the operation of the travelers and air compressors. Alternating current, with a voltage of from 7 200 to 8 200 and an amperage of from 40 to 55, was received from the Astoria Station of the New York and Queens Electric Light and Power Company. It was transformed at the bridge site into direct current with a voltage of from 550 to 600 and an amperage varying from 50 to 900 according to the load, by a 3-phase 60-cycle Allis Chalmers motor generator set, with a capacity of about 500 h.p. Each of the two Ingersoll air compressors was driven by a 75-h.p. electric motor, and had a capacity of 225 cu. ft. of free air



175-TON ARCH TRAVLER

FIG. 39.

compressed per minute to a pressure of at least 135 lb. per sq. in. at the cylinder.

Progress.—The following are the principal dates in connection with the erection of the Arch Bridge (see also Plate XXV):

	Long Island side.	Wards Island side.
Erection of back-stays started........	September, 1914	January, 1915
Erection of arch proper started.....	January, 1915	May, 1915
First six panels erected............	June, 1915	August, 1915
Arch closed........................	October 1st, 1915	
Suspenders and floor system erected..	January, 1916	
Back-stays dismantled...............	April, 1916	
Riveting completed..................	September, 1916	

Contractors' Organization.—The work of the American Bridge Company was under the general charge of C. W. Bryan, M. Am. Soc. C. E., Chief Engineer, and the immediate charge of Mr. C. G. E. Larsson, Assistant Chief Engineer. Mr. J. B. Gemberling, Division Erection Manager, had charge of the erection. The contractor's field organization consisted of a general foreman, three assistant engineers, one foreman, and an average daily force of 140 specially picked men. The maximum force at any one time was 270 men. There were two separate erection gangs, one on each side of the river.

Unusual precautions were taken for the safety of the workmen. Only 5 men lost their lives, mostly due to their own carelessness. Hand ropes and temporary wooden railings were provided on many members, and special steel cages with comfortable platforms were hung from the travelers or from the steelwork for the convenience of the men working on the truss connections (Fig. 33).

12.—Approaches.

The bridge and viaduct approaches to the Hell Gate Bridge consist of 2 735 lin. ft. of steel truss bridges, 10 818 lin. ft. of plate-girder viaducts, and 3 228 lin. ft. of embankment between reinforced concrete retaining walls, with arches over the streets. Except for the bascule spans of the Bronx Kill Bridge, which have open tie flooring, all bridges and viaducts carry ballasted tracks on concrete slabs, weighing 3 500 lb. per lin. ft. of single track. The live load, quality of steel, and permissible unit stresses are given under "Material" and "Loads and Unit Stresses."

Except for the Wards and Long Island Viaducts, which were fabricated and erected by the American Bridge Company in connection with the Hell Gate Bridge, all steel of the bridges and viaducts of the approaches was fabricated and erected by the McClintic-Marshall Construction Company, of which Paul L. Wolfel, M. Am. Soc. C. E., is Chief Engineer and E. A. Gibbs, Assoc. M. Am. Soc. C. E., Manager of Erection. The masonry and foundation work was carried out by the following contractors: T. A. Gillespie Company, Arthur McMullen, Patrick Ryan Construction Corporation, and The Snare and Triest Company. The earthwork was done by the Holbrook, Cabot and Rollins Corporation.

Little Hell Gate Bridge.

The arm of the East River, called Little Hell Gate, which separates Randalls and Wards Islands, is about 1 050 ft. wide at the bridge site. The tracks of the New York Connecting Railroad cross the channel at an angle of about 70° and at a height of 110 ft. above mean high water. The tidal currents are very swift, and navigation, except for small craft, is further obstructed by the shallow depth of the water and the presence of rocks, which, in some places, protrude above the low-water level. In winter considerable quantities of ice are carried through with great force. The rock bottom is in the average only 15 ft. below mean high water.

Although the War Department plans eventually to cut a channel 24 ft. deep and 600 ft. wide through this stream, navigation will probably never be important, and it was not considered necessary to provide a movable bridge for the passage of high-masted vessels. Plans were first made for a bridge 1 050 ft. long, consisting of six deck-truss spans with parallel chords. The spans, which varied in length from 145 to 205 ft., were to rest on solid concrete piers. This design would have secured an economical structure, but it did not meet with the approval of the War Department, which considered the obstruction to the river excessive and the clear head-room above the water insufficient.

In the design finally adopted (Plate XXVII) the number of spans is reduced to four, with an aggregate length of 1 153 ft. 6 in. between centers of abutment piers. The latter piers are square, whereas the three river piers are skew. The two intermediate spans are 296 ft. 6 in. long; the end spans are 280 ft. 2 in. on the center line of the

bridge, or 296 ft. 6 in. on the center line of the longer truss, so that only two different sizes of trusses were required. The deck trusses, two for each span, were made of the bowstring type so as to give additional clear height near the piers for higher vessels which might occasionally pass underneath. The river piers were reduced in size, each to two cylindrical shafts 25 ft. in diameter at the water line, so as to offer the least resistance to the flow of water and ice. A through-truss bridge would have given more clearance, but would have been considerably more expensive on account of the greater width, and would not have harmonized in appearance with the approach viaducts as well as the adopted type. The architecture of the piers and abutments is similar to that of the viaduct piers. The slender, slightly battered river piers, in combination with the graceful outline of the trusses, give the bridge a very pleasing appearance (Fig. 40). The ends of this important structure are befittingly marked by towers extending above the track floor at each abutment.

All foundations are on solid rock, and were placed by open cofferdams. The piers and abutments are of concrete, except that between Elevation 10.7 ft. above mean high water and 1.2 ft. below mean low water, the concrete is protected against disintegration by granite facing.

The two trusses are 52 ft. apart on centers, and have a greatest depth of 50 ft. at the center. The bottom chord has a parabolic shape, and is made up of from ten to thirteen lines of structural-steel eye-bars, 16 in. wide, up to $2\frac{1}{4}$ in. thick, and more than 38 ft. long, connected by 16-in. pins. All other members are riveted. Eye-bars were selected in this case, as they effected a considerable saving over riveted chords. The type of truss is economical, as the chords are of nearly uniform section and the web members are very light. The secondary stresses in the end panels are comparatively large, but those from dead load have been practically eliminated by making the joints and angles between members in accordance with the geometric form of truss and swinging the spans clear of falsework before the connections were riveted. The trusses are connected by a rigid top lateral system and also by stiff sway-frames at each vertical post. No lateral system is provided along the bottom chord, as it was considered preferable to have the wind force, which acts along this chord, transmitted to the lateral system between the stiffer top chords.

Fig. 40.—Little Hell Gate Bridge.

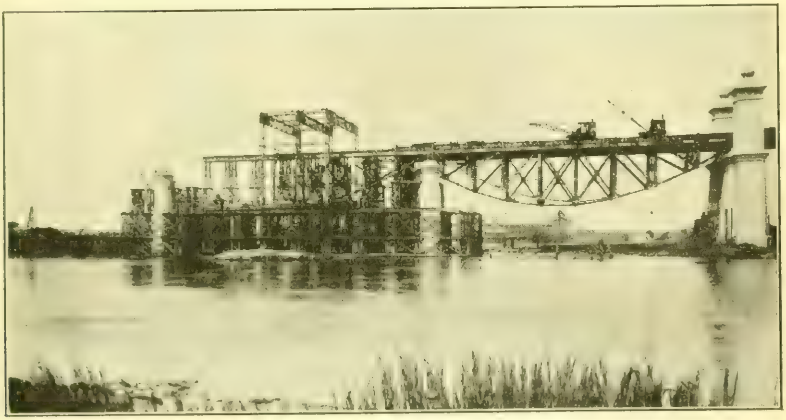
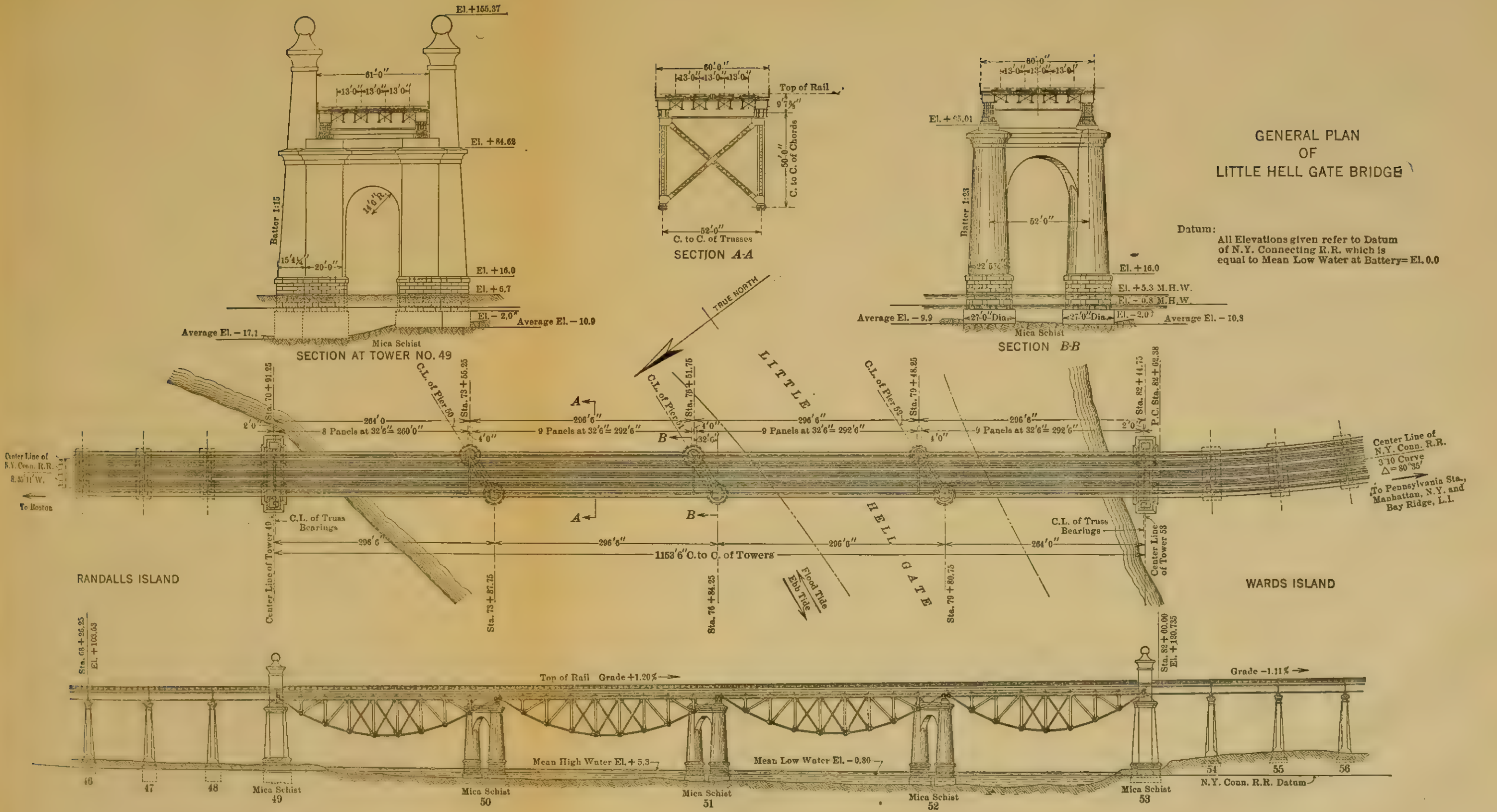


Fig. 41.—Erection of Little Hell Gate Bridge.

PLATE XXVII.
PAPERS, AM. SOC. C. E.
OCTOBER, 1917.
AMMANN ON
HELL GATE BRIDGE.



The two end spans have fixed bearings over the abutments; all bearings over the piers are movable, an expansion joint being provided over the center pier. With this arrangement the piers could be made considerably narrower, as they do not have to resist the longitudinal force from braking. To avoid the transmission of any longitudinal force to the piers through friction, the bearings are provided with cast-steel rockers of the unusual height of 24 in. The relative movement of the two bearings at the center pier is as much as $\pm$ 6 in.

The four spans were erected successively on unusually heavy timber falsework (Fig. 41). A total of 360 000 ft. b.m. of timber was required for each span. Each panel point was supported by a double bent strongly braced transversely and longitudinally. The piles had to be braced above and below water level on account of their small penetration and the strong current of the river. The steel was handled with a steel gantry traveler, which weighed 275 tons, inclusive of equipment. Erection was started in February, and completed in November, 1915.

The bridge contains 25 700 cu. yd. of masonry and 11 250 tons of steelwork. The cost of construction, inclusive of tracks, was approximately $990 000.

Bronx Kill Bridge.

The tracks of the New York Connecting Railroad cross Bronx Kill, which separates Randalls Island from the main land, at an angle of about 72° with the channel and at a height of 75 ft. above mean high water. Bronx Kill is here about 600 ft. wide between shore lines, and 320 ft. between bulkhead lines as established by the War Department. Under present conditions, it is navigable only for small boats. The river bottom consists of various strata of river mud, sand, gravel, and clay overlying the gneiss bed-rock, the surface of which falls from both shores to a maximum depth of about 110 ft. below mean high water under the north abutment. According to plans of the War Department, Bronx Kill is to be improved by dredging a channel 24 ft. deep and 480 ft. wide, principally for the purpose of decreasing the excessive tidal currents in the East River, and incidentally to provide a direct passageway for deep-going vessels between the Harlem River and the upper East River. For this reason the War Department required a movable bridge having two clear openings, of at least 120 ft. each, at right angles to the channel.

The available height of 75 ft. from top of rail to mean high water would have been sufficient for a deck bridge, which would have been more economical than a through structure. The latter type was adopted, however, in order to provide sufficient clear height for all ordinary navigation to pass underneath, and thus restrict the frequency of opening the bridge and the consequent delay to railroad traffic. This consideration was important in view of the prospective heavy traffic, and on account of the heavy grade, on which it is difficult to start trains. Comparative designs were made of various types of movable bridges, including the horizontal draw, the Scherzer rolling lift, and the Strauss trunnion bascule.

The horizontal draw-bridge would have necessitated a very wide pier which would have obstructed the channel excessively and would have required very costly foundations. The Strauss bascule was finally adopted because it was found to be cheapest in first cost, that is, without operating machinery, counterweights, etc., which may not have to be put in for many years. The only parts of the moving mechanism which had to be provided are the trunnions and their bearings, and certain connections for the future attachment of the counterweight trusses.

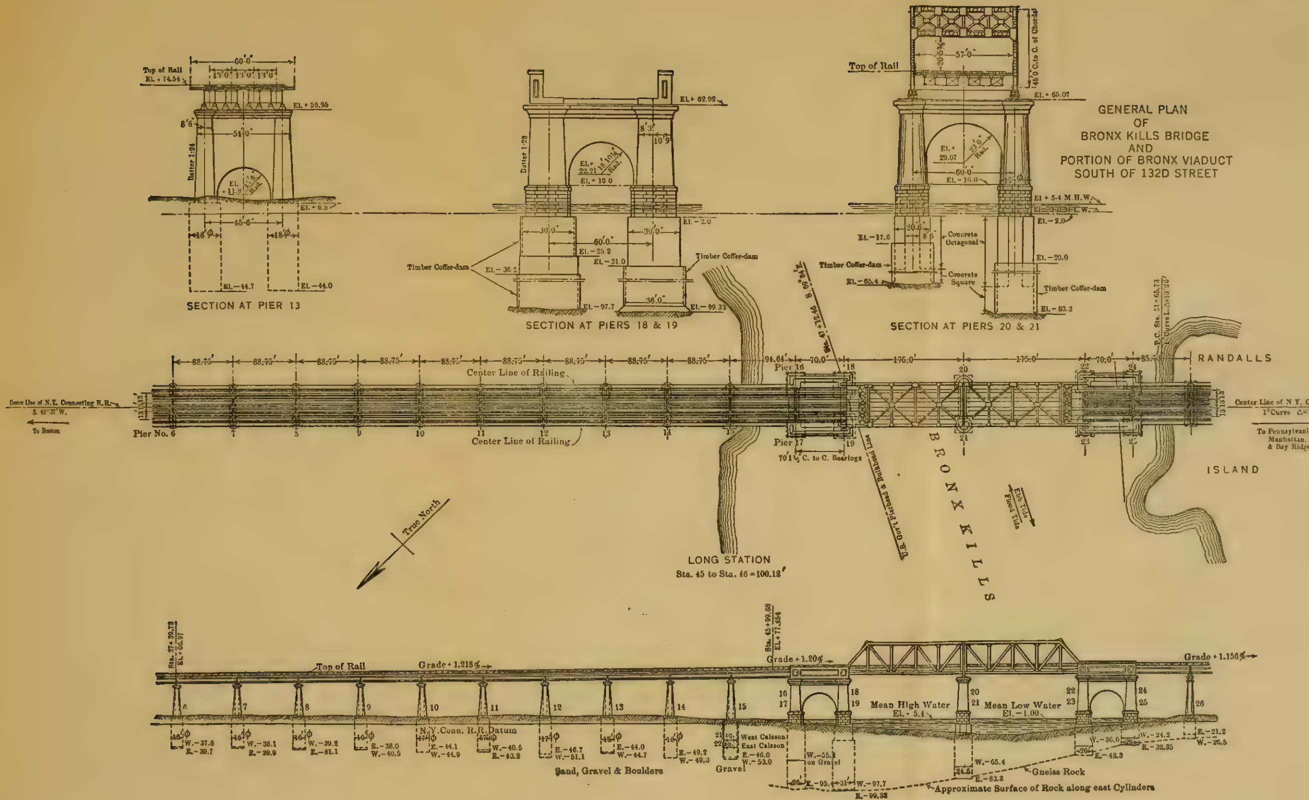
The bridge as built (Plate XXVIII) has two leaves, each 175 ft. long between centers of trunnion bearings and middle pier. Each leaf acts as a single span when closed. All four tracks are carried between the two trusses, which are 60 ft. apart on centers and 45 ft. deep. The trusses are fully riveted.

Each abutment consists of four rectangular piers, connected in pairs longitudinally and transversely by arched concrete walls, and thus forming a rectangular enclosure which will contain the counterweights and hide them from view when the bridge is closed. The two transverse walls support a 70-ft. plate-girder span which now carries the tracks and, ultimately, will carry the operating machinery. The intermediate pier consists of two circular shafts joined by a light concrete arch. The abutments and pier are architecturally treated to conform in appearance to the adjoining viaduct piers (Fig. 42).

All piers were sunk by the pneumatic process, with timber caissons, to depths of from 30 to 105 ft. below mean high water. At the water level the concrete is protected with granite facing in a manner similar to that adopted for the piers of the Little Hell Gate Bridge. The

FIG. 42.—BRONX KILL BRIDGE.

FIG. 43.—CONCRETE ARCH OVER DEBEVOISE (SECOND) AVENUE, LONG ISLAND CITY.



GENERAL PLAN
OF
BRONX KILLS BRIDGE
AND
PORTION OF BRONX VIADUCT
SOUTH OF 132D STREET

steel trunnions, around which the trusses will turn, are of exceptional size, being 30 in. in diameter and 10½ ft. long. The tracks rest on open tie flooring securely fastened to the steel floor system.

The bridge was erected on heavy timber falsework, one span at a time, 256 000 ft. b.m. of lumber having been used in each span. The members were handled with the 50-ton derrick car and a locomotive crane, and these were used for the erection of the adjoining viaducts. Erection was started in August, 1914, and completed in March, 1915. The principal quantities in this bridge are:

Masonry	28 300 cu. yd.
Steelwork:	
Two 70-ft. plate-girder spans...........	470 tons
Two 175-ft. bascule spans (exclusive of machinery and counterweights)....	3 105 tons
Total	3 575 tons

Counterweights, operating machinery, and their supports are estimated to require about 750 tons of steel and 1 300 cu. yd. of concrete filling. The cost, exclusive of counterweights and operating machinery, but inclusive of tracks, was approximately $800 000.

Bronx Viaduct North of 132d Street.

General Conditions.—The design and construction of the portion of The New York Connecting Railroad designated as "Bronx Viaduct north of 132d Street", extending from 132d Street, where the New Haven and the Connecting Roads converge, to the crossing over the New York and Harlem Railroad, about 380 ft. north of 141st Street (Plate XXIX), presented considerable difficulties on account of the unfavorable character of the soil and the proximity of the four New Haven tracks, with a siding along the eastern side of the New York Connecting Railroad from 132d Street to the De La Vergne Machine Works at 137th Street. The two easterly New Haven tracks were originally located alongside the two westerly tracks and, therefore, had to be shifted eastward, to make place for the construction of the two westerly New York Connecting tracks. This had to be done without interruption of traffic on the New Haven tracks. Borings along this entire section indicated a top layer of gravel, coarse sand,

and ashes, from 6 to 12 ft. deep, below this a soft stratum of silt and mud from 12 to 25 ft. deep, resting on a harder stratum of sand and gravel. Only under the easterly tracks near 136th Street does the rock rise to a few feet below the surface.

In 1906, or about 6 years previous to the construction of the Connecting Railroad, the New Haven Railroad had raised its four tracks on an embankment, 65 ft. wide at the top and about 20 ft. above the original ground surface. This embankment consists of cinders, earth, gravel, and a great quantity of large boulders; the latter have gradually settled several feet into the original soft ground and have caused it to heave on the sides. Subsequently, this embankment had to be widened to 95 ft. at the top, when the two easterly tracks were shifted to their present location. The vicinity of this portion of the line is built up mainly with industrial buildings, for which reason no particular consideration had to be given to the architectural treatment of the structure.

Embankment Portion.—On the portion of the Bronx Viaduct between the crossing over the New York and Harlem Railroad and 138th Street, it was found most economical to place the four New York Connecting tracks on fill between reinforced concrete retaining walls, the streets being crossed on deck plate-girder spans. The two retaining walls which enclose the fill under the two westerly tracks—Nos. 5 and 6—rest directly on the New Haven embankment. They are connected by transverse walls, about 20 ft. apart, and thus form, in each block between streets, a monolithic cellular box without bottom. Its maximum height is 19 ft. This box, with the fill between, forms a compact unit which exerts a practically uniform pressure of not more than $1\frac{1}{2}$ tons per sq. ft. on the embankment beneath. Settlements of the retaining walls were expected, but they can have no serious consequences. Since their completion in 1914 a maximum settlement of 8 in. has been observed, without any sign of cracks in the walls. The abutments of the street crossings are independent of the retaining walls, and rest on timber piles. This plan was simple, and proved to be more economical than any other that could have been used. Piles under the retaining walls would have been impracticable on account of the boulder formation of the New Haven embankment, and the expense of any other foundation reaching down to hard soil would have been excessive.

The conditions under the two easterly New York Connecting tracks —Nos. 1 and 2—were different. There, no embankment existed on top of the original surface. The easterly retaining wall under Track No. 1 was carried down to about 7 ft. below the ground surface or 2 ft. below the probable future ground-water level, and was placed on timber piles, from 35 to 45 ft. long, driven to refusal into the hard strata of sand and gravel. The maximum pressure per pile is approximately 20 tons. The westerly retaining wall under Track No. 2 was to have been placed on the new fill between the original New Haven embankment and the retaining wall under Track No. 1. After this fill, which consists largely of sand and loam without boulders, had been placed, it settled considerably, and it was deemed inadvisable to place the wall thereon. Therefore, hollow steel piles, 12 in. in diameter and about 60 ft. long, with cast-steel shoes, were driven through the fill into the hard soil. After being cut off at the proper elevation, the piles were filled with concrete. The wall was then placed on top of these piles. The maximum pressure per pile is 45 tons. To prevent any lateral motion of this wall, due to greater earth pressure on the east side, it is connected to the wall under Track No. 1 with steel tie-rods encased in concrete.

Both retaining walls under Tracks Nos. 1 and 2 are of L-shape and are stiffened by vertical buttresses, 8 ft. apart. They are heavily reinforced with steel rods. This portion of the Bronx Viaduct, about 1 173 ft. long, contains approximately 16 000 cu. yd. of earth fill, 135 000 lin. ft. of timber piles, 8 800 lin. ft. of steel piles, 12 000 cu. yd. of concrete masonry, and 855 tons of structural steelwork. The cost of construction, inclusive of tracks but exclusive of electrification, was about $230 per lin. ft. of 4-track road.

Viaduct Portion.—The embankment type of construction, as previously described, was not practicable south of 138th Street, because, on account of the greater height, the foundation pressures would have become excessive. The plate-girder type of viaduct, with concrete piers, was adopted as being most suitable (Plate XXIX). The spans are generally about 64 ft. long, except over the streets, where they had to be from 72 to 112 ft. The piers are of plain rectangular shape, with simple square copings.

To be safe, the foundations for the piers had to go down through the silt to the hard strata of sand and gravel or rock. Piles would

have been the least expensive foundation, but, in view of the considerable depth of soft, yielding material, they would not have afforded sufficient stability against lateral forces or lateral motion which might be caused by any disturbance of the soft ground in the future construction of adjacent foundations. Furthermore, piles could not have been driven through the boulder formation of the New Haven embankment under the two westerly tracks. Therefore, the more expensive, but securer type of foundation, consisting of two cylindrical concrete caissons under each pier, was adopted. The cylinders are from 10 to 15 ft. in diameter and were sunk partly by open dredging, and partly by the pneumatic process, to depths of from 12 to 55 ft. below the surface. The greatest pressure on the foundations is approximately 8 tons per sq. ft.

A deviation from the foregoing type of viaduct was necessitated at 133d and 132d Streets, where the New Haven and New York Connecting Railroads diverge. On account of insufficient space for piers, it was necessary to frame the longitudinal girders of the two westerly tracks into cross-girders supported by steel columns. The plate-girder viaduct portion, 1 736 ft. long, contains 14 800 cu. yd. of concrete masonry and 7 080 tons of steel work. The cost of construction was about $365 per lin. ft. of 4-track road.

Bronx Viaduct South of 132d Street, Randalls, Wards, and Long Island Viaducts.

The general conditions which affected the design of the viaducts in the Bronx section south of 132d Street, on Randalls and Wards Islands, and on Long Island north of Lawrence Street, are similar, with the exception of the character of the soil. The fact that these four sections are prominently exposed to view called for a uniform and pleasing appearance, in harmony with the monumental character of the arch bridge which they flank on both sides. The type of viaduct adopted consists of deck plate-girder spans resting on arched concrete piers. The span length varies from 72 to 94 ft., except for a single span over Van Alst Avenue in Long Island, which is 130 ft. The lengths chosen are the most economical, except on the Long Island viaduct, where they were largely determined by the location of the many streets which had to be crossed.

On the Bronx Viaduct the piers rest on cylinder caissons, from 15 to 18 ft. in diameter, one under each leg of the pier. These cylinders were sunk partly by open dredging, and partly under air pressure, to depths varying from 45 to 60 ft. below the surface, through soft silt and mud to a hard stratum of sand and gravel or solid rock (Plate XXIX). On all other viaduct sections the piers have ordinary footings built in open excavation. On the Long Island Viaduct, where the soil has various formations of sand, gravel, boulders, loam, and clay, partly saturated with water, which is likely to be drained off, all the footings had to be spread considerably so as to decrease the bearing pressure to the safe value of about 2½ tons per sq. ft., and had to go to depths as great as 34 ft. below the surface.

On Wards and Randalls Islands the piers have comparatively shallow foundations, either on solid rock, or on hard strata of sand and gravel, or hardpan, which permitted pressures of 4 tons per sq. ft. and greater.

The proximity of the railroad to residential districts and to public parks made it desirable to restrict the noise from trains. Embankments between retaining walls were out of the question on account of the great height. Had soil conditions been favorable throughout, a viaduct consisting of a series of solid concrete arches would have been most suitable, as regards appearance and restriction of noise. This type is expensive in first cost, but is more durable, and less costly to maintain, than a plate-girder viaduct, the steel superstructure of which may have to be replaced or strengthened at some future date, and will require frequent painting. Concrete arches, however, require solid, unyielding foundations, in order to prevent dangerous and unsightly cracks, and such foundations could not be had on every section at reasonable cost.

Comparison of Three Designs of Plate-Girder Viaducts.

Fig. 44 shows typical portions of two preliminary designs for the plate-girder viaducts, and the design adopted. Fig. 44(*a*) represents the typical American trestle viaduct with alternate tower spans, 40 ft. long, and intermediate spans of 80 ft. This type, the advantages of which were cheapness and rapidity of erection, is now gradually being displaced by types of greater rigidity and durability, and better appearance. It would have been inappropriate and inadequate for

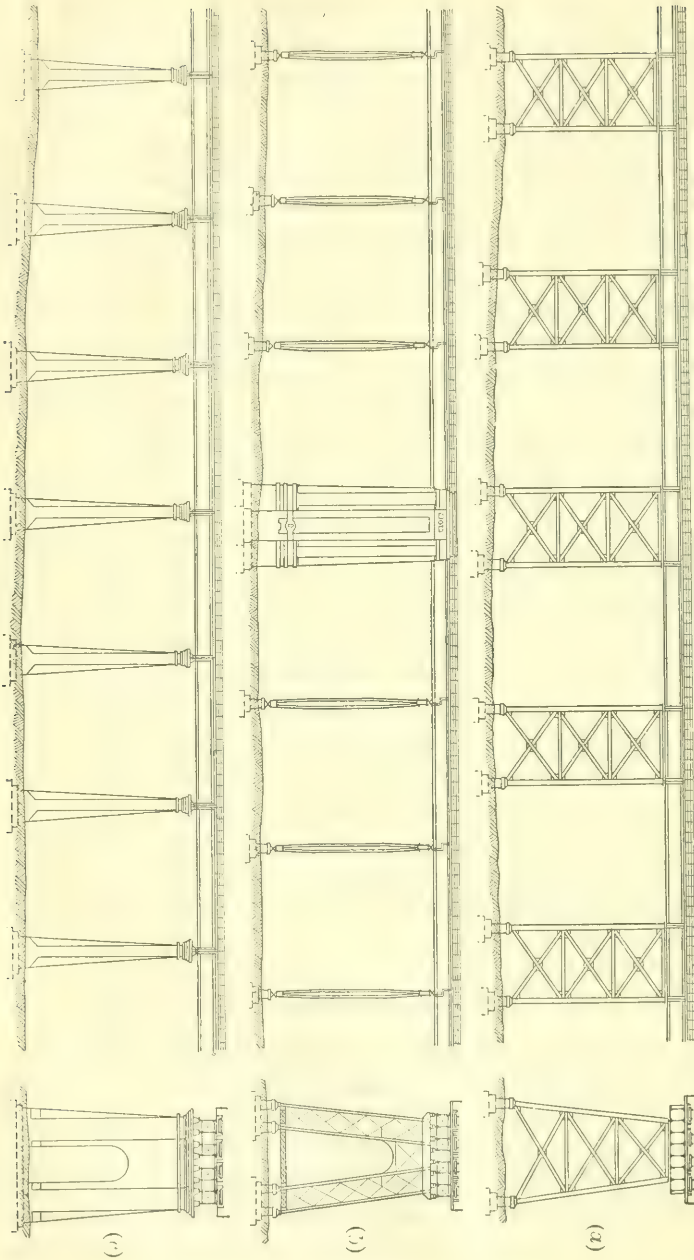


Fig. 44.

COMPARISON OF THREE DESIGNS OF PLATE GIRDER VIADUCTS

the approaches to the Hell Gate Bridge. Fig. 44(b) represents the design made by Mr. Lindenthal in 1906. It consists of plate girders, of nearly uniform span length of from 70 to 80 ft., resting on steel rocker bents. To resist the longitudinal forces from braking and traction solid masonry piers (stability piers) were to be provided at about every tenth span. This design is superior in general appearance to the trestle design. The stability piers convey the impression of rigidity, and give opportunity for architectural treatment. The arch form selected for the steel rocker bents, although somewhat more expensive, is more pleasing than the ordinary two-column bent with single intersection diagonals. This type of viaduct is also stiffer, in the longitudinal direction at least, than the trestle type. Fig. 44(c) represents the type finally adopted, with concrete piers. It is superior to the other two types in appearance, rigidity, and durability, and is less costly to maintain.

A comparison of estimated costs, with the prices prevailing at the time the design was made, showed that, for an average height of viaduct of 100 ft. on tangent, the steel trestle design would have been about 20% cheaper, and the design with steel rocker bents about 10% cheaper, than the adopted design. On a 3° curve the saving in first cost would have been only 15% and 5%, respectively, as the centrifugal force of the trains requires additional material in the steel bents and towers, but not in the masonry piers. For heights of viaducts of less than 100 ft., the differences in cost are correspondingly less. With the high prices of steel prevailing at present, there would be little, if any, saving in favor of the steel trestle type.

The arched concrete piers mark a radical departure from the ordinary solid square concrete piers with plain surface and simple square coping. The rectangular body of the pier proper is only 6 ft. thick, from the coping down, but is reinforced by four buttresses which have a batter of 1 : 15. These piers convey the impression of elegance and yet of great rigidity. The appearance is enhanced by the massive and architecturally elaborate coping of cornices and mouldings.

The concrete is made of 1 part Portland cement, 2 parts sand and 4 parts gravel or broken stone, and is reinforced with steel rods, vertically and horizontally, against shrinkage and temperature cracks.

The plate-girder spans, typical details of which are shown in

Plate XXX, present no unusual features, except for the cast-steel bearings, which were designed to keep the reaction from one-sided loading as close as possible to the center of the pier. Special attention was also given to efficient web splices.

Erection of Plate-Girder Viaducts.

The erection of the viaducts presented no unusual difficulties, and the great number of nearly uniform spans, with the large tonnage involved, afforded opportunity for economical and rapid erection. The Bronx and Randalls Island Viaducts, for which the McClintic-Marshall Construction Company had the contract, were erected by a 50-ton steel derrick car, in some operations assisted by a 50-ton locomotive crane (Fig. 45).

All material for these viaducts was delivered over temporary tracks laid on the finished portion of the viaduct. Where possible, the girders were shipped and erected riveted up in pairs at the shop. A remarkable record was made on March 8th, 1915, when, after careful preparation, twenty-two single-track spans, with an aggregate weight of 1 504 tons, were put in place in a single 8-hour day.

A somewhat different method was used in the erection of the Wards and Long Island Viaducts, for which the American Bridge Company had the contract. After the Hell Gate Arch had been closed, and as the temporary back-stays were being dismantled, the plate girders, about 50% of which had formed part of the back-stays and counterweights, were distributed on the ground along the viaducts by using a locomotive crane running on a temporary track. The two 65-ton steel travelers, which had previously been used for the erection and dismantling of the back-stays of the arch bridge, were set up at the ends of the viaducts and proceeded toward the Hell Gate Bridge, raising the girders singly from the ground (Fig. 46).

Quantities, Weights, and Cost of Viaducts.

Table 6 gives the principal dimensions and quantities and the cost per linear foot for the different viaduct sections.

The weight of the steelwork (exclusive of I-beams in flooring), in pounds per linear foot, of single-track plate-girder spans, can be expressed approximately by the formula:

$$W = 350 - 17\ l$$

for spans lengths of $l = 72$ to 94 ft.

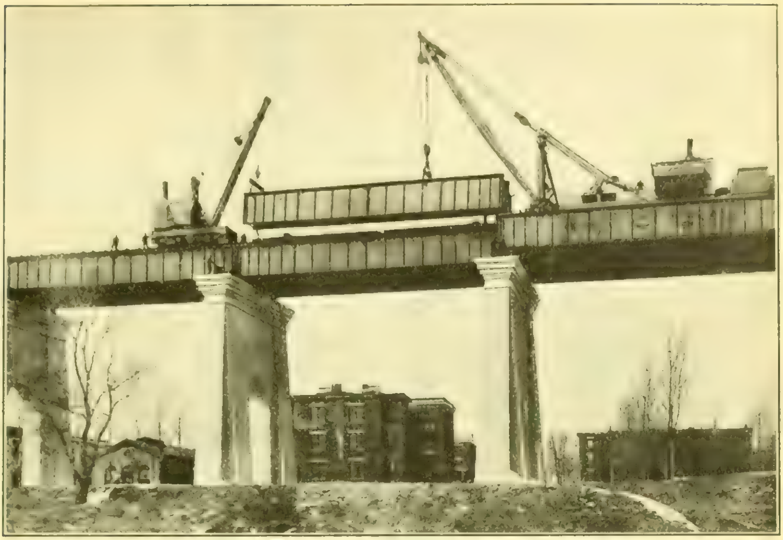


FIG. 45.—ERECTION OF RANDALLS ISLAND VIADUCT.

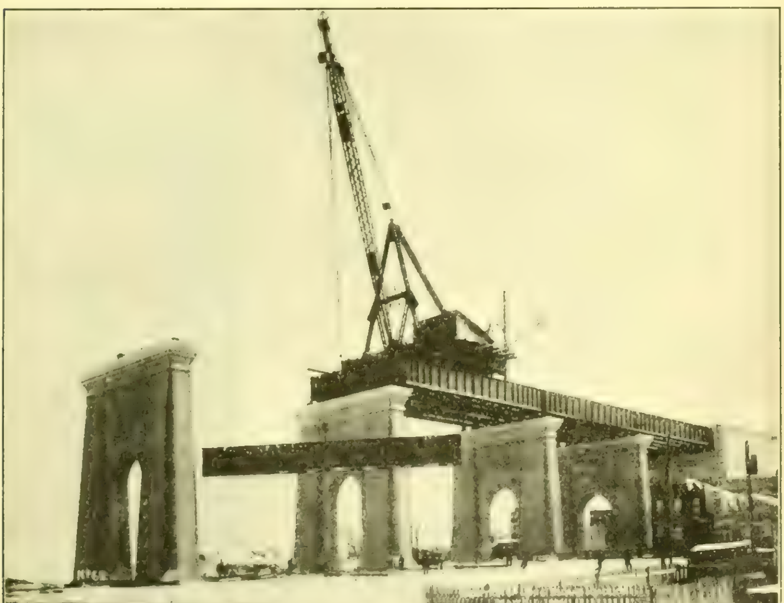
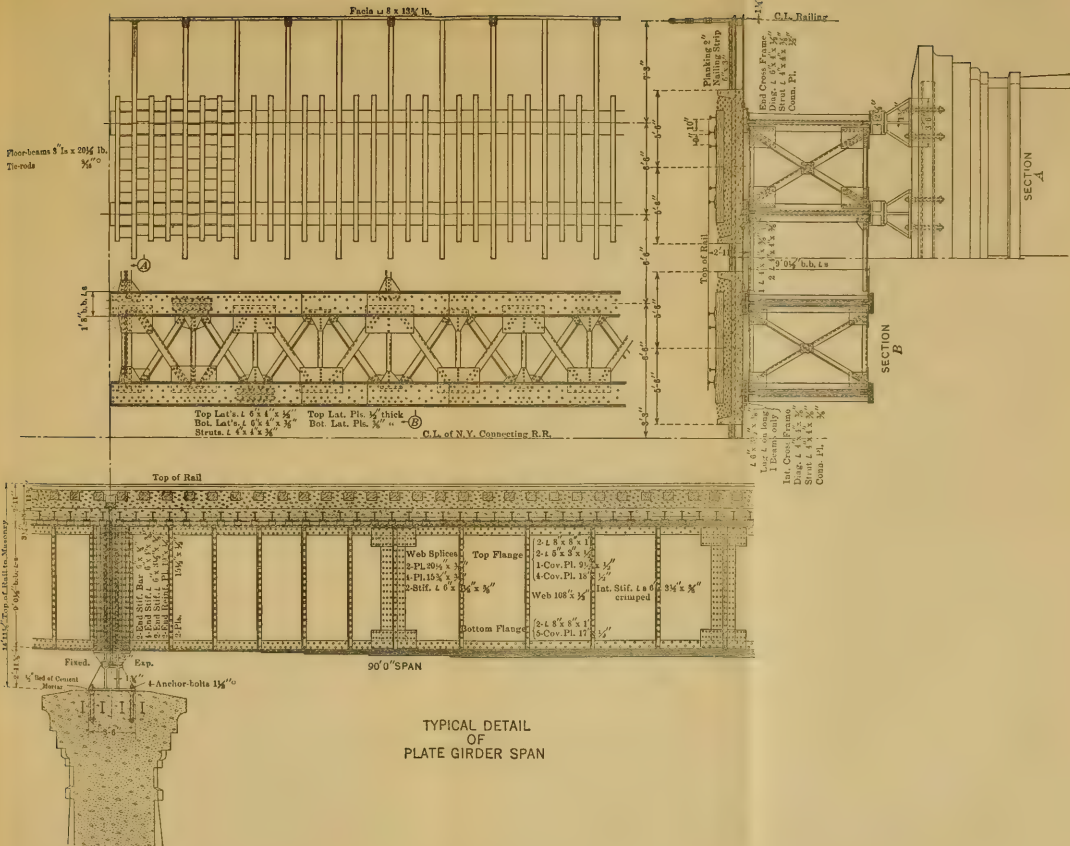


FIG. 46.—ERECTION OF LONG ISLAND VIADUCT.



TYPICAL DETAIL
OF
PLATE GIRDER SPAN

TABLE 6.—DIMENSIONS, QUANTITIES, COST, ETC., OF VIADUCTS.

Section.	Bronx Viaduct South of 132d St.	Randalls Island Viaduct.	Wards Island Viaduct.	Long Island Viaduct.
Length of section, in feet	1 071	1 965	2 654	2 868
Approximate average height of rail above ground surface, in feet	60	80	110	90
Type of foundation	Caissons 45 to 60 feet deep.	Mostly shallow and narrow footings.	Mostly shallow and narrow footings.	Mostly deep and wide footings.
Average span length, in feet	89.0	82.0	88.5	85.5
Total quantity of concrete in piers, in cubic yards	17 500	29 800	60 300	77 900
Total weight of steelwork, in tons	4 375	7 610	11 190	11 980
Approximate cost per linear foot of viaduct, inclusive of tracks	$445	$370	$445	$500

Deck Truss Bridges of Long Island Eastern Viaduct.

Potter Avenue, and the streets between Steinway and Flushing Avenues, in Long Island City, are crossed by deck-truss bridges aggregating 1 231 ft. in length. On account of excessive span length and limited height, concrete arches, such as were built over the other streets on the Long Island Eastern Viaduct, would have been impracticable. Potter Avenue, which is 80 ft. wide, is crossed by the railroad at an angle of only 19 degrees. The distance between street lines in the direction of the tracks is about 250 ft., and the rails are only 50 ft. above street level. A skew steel bridge, about 270 ft. long, with abutments parallel to the street lines, would have been a very unsatisfactory design, and probably as expensive as the one adopted, although lighter in steelwork.

The bridge, as built, consists of three square deck-truss spans, each 135 ft. long (Plate XXXI and Fig. 47). The two ends are supported on concrete abutments, which form a monolithic structure with the embankment side-walls. Two heavy steel rocker bents, each consisting of two columns and a cross-girder, form the intermediate supports. Two of the columns are placed in the center of the street, on permission secured from the city. Each span has four trusses, one for each track. They are $18\frac{1}{2}$ ft. deep, and 13 ft. 9 in. apart on centers. The trusses have fixed bearings on one abutment and expansion bearings on the other, the intermediate steel bents acting as rockers. The longitudinal forces from all spans, therefore, is transmitted to one end, and all temperature expansion takes place toward

the other end. Over the steel bents the trusses are supported at their top chord points by cast-steel pin bearings, which in turn are supported by the cross-girders of the bent. Each of two adjoining trusses turns independently on its bearing, and the bottom chord member opposite the bearing is free to slide at its end, so that the trusses act as simple spans.

The columns are provided with pin bearings at the bottom, so as to be free from bending stresses from longitudinal forces or temperature expansion of the trusses. Transversely, however, the columns, with the cross-girders, form rigid portals which transmit the reactions from the lateral forces to the foundations. The cross-girders are double web-girders, 10 ft. deep, and weigh 130 tons each. They were shipped in three sections. The floor-beams rest on top of the trusses, and the stringers are framed into them. To avoid stresses in the floor-beams, due to unequal deflections of the trusses, all floor-beams except those over supports are interrupted between the two interior trusses.

The design of the truss bridges, between Steinway and Flushing Avenues, was governed by similar conditions, and is similar in every respect to that of the Potter Avenue Crossing, except that all intermediate supports are solid concrete piers instead of steel bents, and are on railroad property. The spans vary in length from 120 ft. to 165 ft. 11 in. The depth of the trusses is only 16 ft. 4 in., which was limited by the required minimum height of 16 ft. above the street level. The trusses, therefore, are unusually heavy.

All these truss bridges were erected on heavy timber bents by an 80-ton steel traveler (Fig. 48). Erection was started in October, 1914, and completed in April, 1915. Potter Avenue Crossing contains approximately 4 000 cu. yd. of concrete masonry and 3 722 tons of steel work, and the cost of construction, inclusive of tracks, was about $275 000, or $675 per lin. ft. The Steinway-Flushing Avenue Crossing contains approximately 7 600 cu. yd. of masonry and 6 526 tons of steelwork, and its cost of construction, inclusive of tracks, was about $500 000, or $565 per lin. ft.

Embankment Portion of Long Island Eastern Viaduct.

Except for the steel truss bridges over certain streets, as described before, the Eastern Viaduct, which extends from Lawrence Street to Stemler Street in Long Island City, a total length of 3 500 ft.,

FIG. 47.—POTTER AVENUE CROSSING.

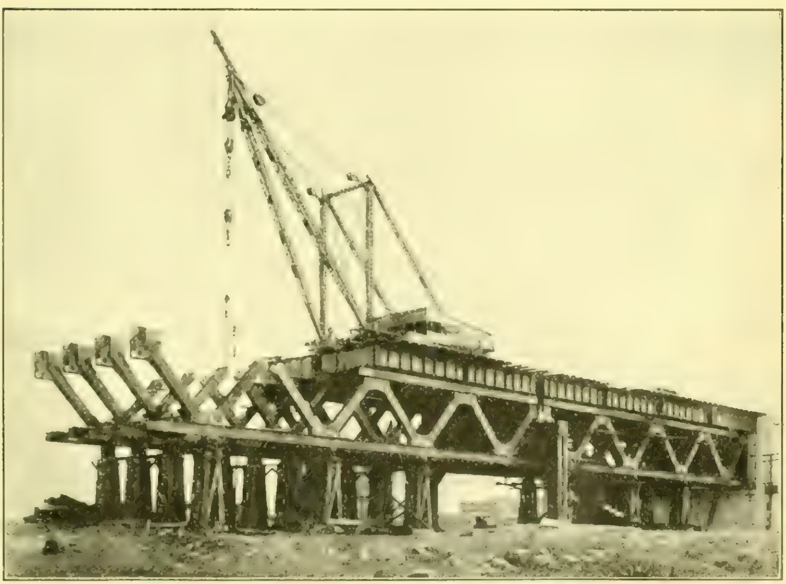


FIG. 48.—ERECTION OF POTTER AVENUE CROSSING.

consists of a novel type of embankment, from 30 to 65 ft. in height above ground. Seven streets are crossed by reinforced concrete arches which form a monolithic structure with the retaining walls of the embankment. (Plate XXXI and Fig. 43.)

The embankment consists of two longitudinal reinforced concrete retaining walls, connected and held in relative position by horizontal steel tie-rods, which are embedded individually in a shell of concrete for protection against corrosion. These rods resist the pressure from the earth fill. For additional stability, the two walls are connected by thin cross-walls, about 50 ft. apart. The arches consist of a comparatively thin barrel reinforced by vertical ribs. The fill is mixed clay, sand, and gravel, carefully placed in 12-in. crowned layers, and thoroughly tamped, so as to form a uniform compact mass which exerts a comparatively small pressure on the retaining walls. It is thoroughly drained by chimneys of rock packing which extend along the walls from the top of the fill to the weep-holes at the bottom.

The walls and arches have perfectly plain surfaces and a simple coping. No attempt has been made at architectural treatment, because the territory in the vicinity is being built up mostly by industrial buildings which hide that portion of the railroad from prominent view. This embankment construction is considerably cheaper than the ordinary type, which consists of a fill between two independent gravity walls. For a height of 50 ft. the latter type would have cost from 30 to 40% more.

A plate-girder viaduct with concrete piers, such as was used north of Lawrence Street and over the island, would also have been more expensive than the adopted type of embankment. For heights exceeding about 65 ft., however, the plate-girder viaduct became cheaper.

The embankment contains approximately 70 000 cu. yd. of concrete masonry, 160 000 cu. yd. of earth fill, and 1 000 tons of steel reinforcement.

The cost of construction per linear foot of embankment, inclusive of tracks, was approximately $470 for a height of 65 ft. and $280 for a height of 35 ft., or ($6.5\ h - 50$) dollars per square foot of elevation, h being the height of rail above the ground line.

13.—Track Floor Construction.

The franchise required a ballasted roadbed on the Bronx Section, north of Bronx Kill, and on the Long Island Section, south of Hell

Gate. A solid ballasted floor, owing to its advantages over the open tie floor, namely, more uniform roadbed, less noise and impact, smaller cost of maintenance, and greater safety in case of derailment or fire, was adopted throughout, however, except for the bascule spans of the Bronx Kill Bridge, although it involved a considerable additional initial expenditure over the open tie floor.

Various types of solid floor construction were considered. Previous to construction a wooden floor, consisting of framed, treated ties, laid closely together, appeared to be most suitable, owing to its lightness and, as it seemed then, its comparatively small cost. By the time construction started, however, the prices of framed and treated timber had increased considerably, and a more durable and fireproof, although somewhat heavier and slightly more expensive, concrete floor construction was adopted.

A reinforced concrete slab, in which the compressive strength of the concrete and the bond between the steel rods and the concrete are relied on to resist the stresses, was not considered advisable, in view of the limited experience as regards the durability of such a construction under the heavy impact to which the floor would be subjected.

The type finally adopted (Fig. 49) consists of 8-in. I-beams, about 15 in. from center to center, placed across the two stringers or girders of each track. These beams are tied together near the bottom with $\frac{5}{8}$-in. tie-rods, about 10 in. apart, and thus form, by themselves, a comparatively rigid steel skeleton. The axle load of 70 000 lb., plus 200% impact, was assumed as distributed over three beams, or an equivalent length of track of twice the height from top of rail to top of I-beams.

The concrete, which is placed between and from $2\frac{3}{4}$ to $3\frac{1}{2}$ in. above the beams, but flush with the bottom surface of the beams, forms merely an encasement. To prevent transverse temperature cracks at the top surfaces, the concrete is further reinforced by longitudinal rods resting on, and tied to, the steel beams. The ballast is held on each side of the track by parapet walls, which are also well reinforced against temperature or shrinkage cracks. The I-beams served as ties for the construction tracks, and thereby saved the cost of temporary timber ties. Plain steel sheets, $\frac{1}{8}$ in. thick, wedged against the bottom of the beams, constituted simple forms for the bottom surface of the concrete. The space between the concrete troughs is utilized for

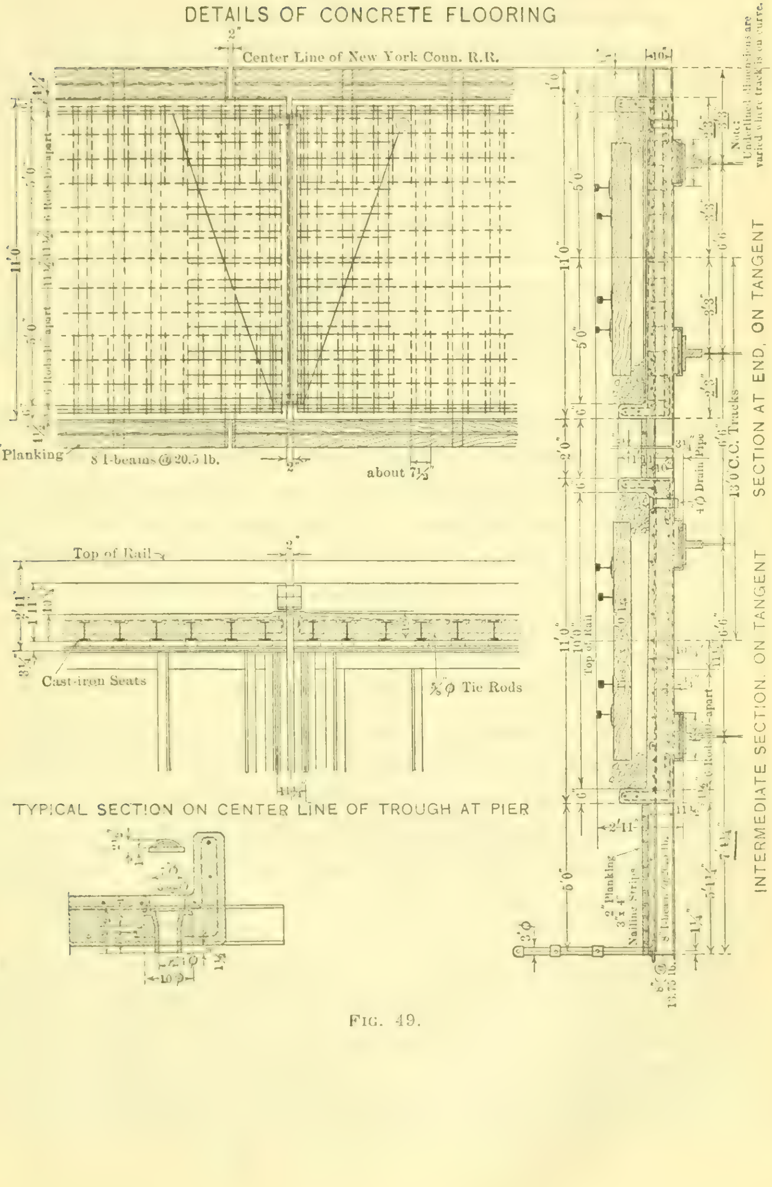

DETAILS OF CONCRETE FLOORING
Center Line of New York Conn. R.R.
2" Planking
8 I-beams @ 20.5 lb.
about 7½"
Top of Rail
Cast-iron Seats
⅝ φ Tie Rods
TYPICAL SECTION ON CENTER LINE OF TROUGH AT PIER
SECTION AT END, ON TANGENT
INTERMEDIATE SECTION, ON TANGENT
13'0" C.C. Tracks
4" φ Drain Pipe
2" Planking
3"x 4" Nailing Strips

FIG. 49.

footwalks and for the six-duct concrete conduit construction. Expansion joints in the concrete troughs are provided at all expansion joints in the steel superstructure. These joints are covered by T-shaped steel dams, bedded on a thin layer of asphalt.

Care was taken to secure a dense concrete by using a mixture of 1 part Portland cement, 2 parts of well-graded sand, and 4 parts of broken limestone, the latter composed of 75% of ¾-in. stone and 25% of screenings. This concrete, tested on 4-in. cubes, showed an average compressive strength of about 3 500 lb. per sq. in. at the age of 28 days. The top of the concrete slab was carefully troweled to a smooth finish. No water-proofing material was placed thereon.

For efficient drainage, the top surface was given a transverse slope of 1½ in. in 10 ft., and 4-in. cast-iron drain pipes, with strainers, were placed about 15 ft. apart. On the street crossings, these pipes discharge into steel gutters which lead to 6-in. down-spouts at the piers or abutments.

The sidewalks and the walks between the tracks are of 2-in. wooden planks resting on extensions of the I-beams. The flooring contains, per linear foot of 4-track structure, 1.5 cu. yd. of concrete, 1 200 lb. of steel, and 40 ft. b. m. of timber, and cost about $53 per lin. ft. The I-beams were furnished and erected by the contractors for the steelwork. The concrete and the timber flooring were placed on contract by Fraser, Brace and Company and The Snare and Triest Company.

14.—Engineering Organization.

The New York Connecting Railroad has been built under the direction of Mr. Samuel Rea as President and Mr. A. T. County, Assistant to the President. Mr. Gustav Lindenthal, Consulting Engineer and Architect, prepared the plans for the East River Bridge Division, and, as Chief Engineer, directed their execution. During construction the Chief Engineer was assisted by an engineering staff of ninety-five members.

O. H. Ammann, M. Am. Soc. C. E., Assistant Chief Engineer, had general charge of the office, field, and inspection work, H. W. Hudson, M. Am. Soc. C. E., Construction Engineer, was in direct charge of the field operations, in which he was assisted, in the earlier stages of the work, by three Resident Engineers, George W. Philips, Assoc. M. Am. Soc. C. E., R. T. Robinson, Assoc. M. Am. Soc. C. E., and S. D.

Reed, Assoc. M. Am. Soc. C. E., and later by Mr. S. D. Reed as Assistant Construction Engineer. D. B. Steinman, Assoc. M. Am. Soc. C. E., Special Assistant Engineer, attended to computations and strain measurements, and Mr. W. A. Cuenot, Assistant Engineer, to the drafting and checking of the plans and shop drawings.

A very thorough inspection was exercised over all materials. All cement which went into the work was tested at the Company's laboratory, in charge of Mr. G. B. MacWhinney, Assistant Engineer. The steel was tested and inspected at the mills and at the various shops by a corps of fifteen inspectors, directed successively by Mr. J. C. Naegeley, as Engineer of Inspection, and Messrs. William E. Crane and R. E. McGough, as Chief Inspectors.

Conclusion.—Some of the broader engineering questions which suggest themselves in the design and execution of the structure forming the subject of this paper may be summarized as follows:

A great engineering work cannot be spontaneously created in its final, perfect form, but has to grow and develop gradually, in its entirety as well as in its constituent parts. Although the layman can only judge such a work in the light of an accomplished fact, the engineer must ever be conscious that it is only through extensive and laborious preliminary studies, and untiring efforts to improve, that he can hope to achieve a perfect work.

In the execution of a great and complex engineering or scientific undertaking collaboration of experts in various fields is essential, but a great structure of monumental character must be the product of an individual creative and directive mind.

A great structure cannot be the result of a set of rules and specifications, nor of elaborate mathematical computations. Such a work requires wide experience and sound judgment, and therefore, should be entrusted only to engineers of high professional attainments and reputation.

Throughout this paper the importance of rigidity in bridge construction has been pointed out. Rigidity insures greater durability and safety. There are remarkable examples of structures which have stood up under excessive strains under which they would have failed had it not been for the rigidity of their members or connections. Large bridges must be built for generations to come. Engineers to-day cannot afford to build important structures cheaply, to serve their

purpose for the time being, and incur the risk of having to replace them after a short period of usefulness.

Emphasis has been laid on the appearance of the structures described. Engineering structures are still regarded by many engineers as mere works of utility, which deserve no consideration in architectural or artistic treatment. So long as this opinion prevails, the Engineering Profession will not lift itself to a higher plane, and it is even running the risk of being relegated to second place—or after the architect—in the creation of such monumental structures as properly belong in its domain.

APPENDIX A

Calculation of Dead-Load, Live-Load, and Temperature Stresses in Arch Trusses of Hell Gate Bridge.

1.—Influence Line for Horizontal Reaction of 2-Hinged Arch.

In calculating the influence line for horizontal reaction, the following analytical method has been applied:

A.—Calculation of Values of Δs, $\Delta \alpha$, and Δl.— If the arch is considered a simple span fixed at A and free to move at B (Fig. 50) and if, for any one member of the truss,

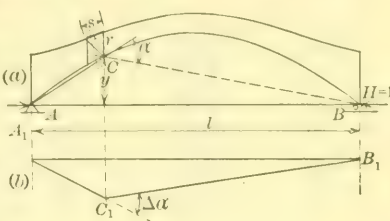


FIG. 50.

A = area of its gross section, in square inches;

s = its length, in feet;

r = its lever arm (perpendicular distance from its center of moments, C), in feet;

y = ordinate of its center of moments, above line connecting hinges, in feet;

$S_1 = \dfrac{y}{r}$ = stress in the member due to the sole application of a horizontal force of unity at B, and E = modulus of elasticity (30 000 000 lb. per sq. in.); then the axial deformation of the member is, in feet,

$$\Delta s = \frac{S_1 s}{A E} \qquad (1)$$

If it is assumed that only this one member is elastic, the angle, α, between the lines, $A\ C$ and $B\ C$, will change by an amount

$$\Delta \alpha = \frac{\Delta s}{r} \text{ (arc measure)} \qquad (2)$$

that is, the elastic line is a triangle, $A_1\ C_1\ B_1$ (Fig. 50 (b)) the sides of which, $A_1\ C_1$ and $B_1\ C_1$, form the angle $\Delta \alpha$.

The point, B, moves horizontally, that is, the span length, l, changes by an amount, in feet, equal to

$$\Delta l = \frac{y}{r} \Delta s = y \Delta \alpha \qquad (3)$$

The sum, $\Sigma \Delta l$, of the values Δl, for all truss members gives the total horizontal movement of the point, B, due to the sole application of the horizontal load of unity at B.

Tables 7 and 8 show the calculation of the values, Δs, $\Delta \alpha$, and Δl, for each truss member, and also the sum, $\Sigma \Delta l$. For convenience

TABLE 7.—DETERMINATION OF VALUES

	Member.	(1) Length, s, in feet.	(2) Gross Area, A, in square inches.	(3) Lever Arm, r, in feet.	(4) Ordinate of Center of Moments, y, in feet.	(5) Stress $S_1 = \frac{y}{r}$
BOTTOM CHORD.	0– 2	56.131	1 384	— 106.00	+ 140.00	— 1.3208
	2– 4	54.012	1 281	— 88.86	+ 149.60	— 1.6835
	4– 6	52.022	1 265	— 75.92	+ 162.93	— 2.1461
	6– 8	50.173	1 337	— 67.76	+ 180.00	— 2.6562
	8–10	48.484	1 163	— 62.38	+ 197.83	— 3.1714
	10–12	46.971	1 121	— 57.39	+ 213.43	— 3.7180
	12–14	45.651	1 080	— 52.88	+ 226.80	— 4.2890
	14–16	44.542	1 045	— 48.92	+ 237.94	— 4.8639
	16–18	43.661	988	— 45.60	+ 246.85	— 5.4134
	18–20	43.020	946	— 43.01	+ 253.54	— 5.8949
	20–22	42.630	929	— 41.21	+ 258.00	— 6.2606
	22–22 A	42.500	929	— 40.22	+ 260.22	— 6.5669 *
TOP CHORD.	1– 3	43.571	315	+ 110.16	+ 36.67	+ 0.3328
	3– 5	44.541	315	+ 88.67	+ 70.00	+ 0.7814
	5– 7	45.800	315	+ 74.24	+ 100.00	+ 1.3470
	7– 9	46.089	329	+ 65.62	+ 126.67	+ 1.9303
	9–11	45.273	385	+ 59.54	+ 150.00	+ 2.5193
	11–13	44.553	385	+ 54.18	+ 170.00	+ 3.1377
	13–15	43.936	385	+ 49.60	+ 186.67	+ 3.7634
	15–17	43.424	385	+ 45.85	+ 200.00	+ 4.3651
	17–19	43.023	385	+ 43.01	+ 210.00	+ 4.8826
	19–21	42 733	385	+ 41.11	+ 216.67	+ 5.2704
	21–23	42.558	329	+ 40.16	+ 220.00	+ 5.4781
	23–23 A	42.500	329	+ 40.22	+ 220.00	+ 5.3729 *

in calculation, the foregoing values have been determined in units of 1 000 E. (See Columns 6, 7, 8, and 9 of Tables 7 and 8.)

B.—Determination of Elastic Curve of Arch Truss.—As can easily be proved, the elastic line, A_1 C_1 B_1 (Fig. 50 (b)), assuming again only the one member elastic, is identical with the moment diagram of a simple span, A B, due to the sole application of a vertical load, $\Delta\ \alpha$, at the center of moments, C, of the member in question.

From this rule, if applied to every truss member, it follows that the elastic line of the arch truss due to the sole application of a horizontal load of unity at B is identical with the moment diagram due to the application of the values, $\Delta\ \alpha$, as vertical loads, called "Elastic Loads", at the respective centers of moments of the truss members.

The center of moments of a chord member, and therefore the corresponding elastic load, $\Delta\ \alpha$, is always at a panel point.

The center of moments of a web member, in general, is not at a panel point, nor is it always within the span length, and, therefore, it

Δs, $\Delta \alpha$, and Δl, FOR CHORD MEMBERS.

(6) $1\,000\, E\, \Delta s = 1\,000\, \frac{S_1\, s}{A}$	(7) $1\,000\, E\, \Delta \alpha = 1\,000\, \frac{E\, \Delta s}{r}$	(8) $1\,000\, E\, \Delta l = \frac{y}{r}\, 1\,000\, E\, \Delta s = y\, 1\,000\, E\, \Delta \alpha$
− 53.57	+ 0.505	+ 70.7
− 70.98	+ 0.799	+ 119.5
− 88.26	+ 1.163	+ 189.4
− 99.68	+ 1.471	+ 264.8
− 132.21	+ 2.119	+ 419.3
− 156.19	+ 2.715	+ 579.5
− 181.29	+ 3.428	+ 777.6
− 206.90	+ 4.229	+ 1 006.3
− 239.22	+ 5.246	+ 1 295.0
− 268.08	+ 6.283	+ 1 580.3
− 287.29	+ 6.971	+ 1 798.6
− 300.42	+ 2 × 3.735	+ 2 × 971.9
	$1\,000\, E\, \frac{\Sigma \Delta l}{2} =$	+ 9 071.9
$1\,000\, E\, \Sigma \Delta l$ for bottom chords =		+ 18 143.8
+ 46.04	+ 0.418	+ 15.3
+ 111.63	+ 1.259	+ 88.1
+ 195.85	+ 2.638	+ 263.8
+ 270.41	+ 4.121	+ 522.0
+ 296.25	+ 4.976	+ 746.4
+ 363.10	+ 6.702	+ 1 139.3
+ 429.48	+ 8.659	+ 1 616.3
+ 491.99	+ 10.731	+ 2 146.1
+ 545.62	+ 12.686	+ 2 664.0
+ 584.99	+ 14.230	+ 3 083.2
+ 708.62	+ 17.645	+ 3 881.9
+ 693.95	+ 2 × 8.627	+ 2 × 1 897.5
	$1\,000\, E\, \frac{\Sigma \Delta l}{2} =$	+ 18 063.9
$1\,000\, E\, \Sigma \Delta l$ for top chords =		+ 36 127.8

(9) Notes.

* Stresses corrected for effect of double diagonals in center panel.

For Member 22-22 A:

$\frac{y}{r}$	$= -6.4699$
Correction ($H = 1, P = 0$)	$= -0.0970$
Stress S_1	$= -6.5669$

For Member 23-23 A:

$\frac{y}{r}$	$= +5.4699$
Correction ($H = 1, P = 0$)	$= -0.0970$
Stress S_1	$= +5.3729$

is more convenient to substitute for the load, $\Delta \alpha$, applied at the center of moments of a web member, two vertical panel loads as follows:

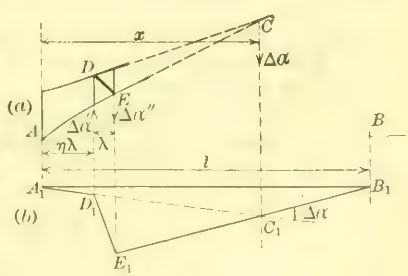


FIG. 51.

Take, for instance, the diagonal, $D\,E$ (Fig. 51), the center of moments of which is at C. The elastic line, considering only $D\,E$ elastic, is a broken line, $A_1\, D_1\, E_1\, B_1$, the segments of which, $A_1\, D_1$ and $B_1\, E_1$, intersect at C_1 vertically below C, and enclose the angle, $\Delta \alpha$. It can easily be proved that this line is identical with the moment diagram due to the vertical loads, $\Delta \alpha' = \Delta \alpha \left[\frac{x}{\lambda} - (n + 1)\right]$ and $\Delta \alpha'' = \Delta \alpha \left(\frac{x}{\lambda} - n\right)$, applied at the panel points, D and E,

TABLE 8.—DETERMINATION OF VALUES,

	Member.	(1) Length, s, in feet.	(2) Gross area, A, in square inches.	(3) Lever arm, r, in feet.	(4) Ordinate of center of moments. y, in feet.	(5) Stress, $S_1 = \frac{y}{r}$
DIAGONALS.	1- 2	111.732	235.5	— 222.2	+ 189.6	— 0.8535
	3- 4	90.235	228.4	— 247.2	+ 224.9	— 0.9097
	5- 6	75.937	201.8	— 321.8	+ 285.6	— 0.8875
	7- 8	68.197	129.8	— 401.5	+ 341.4	— 0.8504
	9-10	63.984	129.8	— 387.7	+ 341.4	— 0.8805
	11-12	60.765	129.8	— 380.1	+ 341.3	— 0.8981
	13-14	58.454	129.8	— 383.1	+ 341.3	— 0.8907
	15-16	56.971	168.8	— 405.2	+ 341.2	— 0.8422
	17-18	56.251	168.8	— 465.6	+ 341.5	— 0.7335
	19-20	56.266	196.0	— 615.9	+ 341.5	— 0.5545
	21-22	57.011	196.0	— 1 113.6	+ 340.5	— 0.3050
	23-22	58.514	129.8			+ 0.1335*
VERTICALS.	0- 1	140.000	312.0 } Average	+ 42.5	+ 36.7	+ 0.8628
	2- 3	112.933	255.0 } Average	+ 202.2	+ 195.3	+ 0.9656
	4- 5	92.930	235.0 } Average	+ 237.3	+ 237.3	+ 1.0013
	6- 7	80.000	151.0	+ 354.3	+ 322.3	+ 0.9097
	8- 9	71.163	137.0	+ 549.6	+ 428.4	+ 0.7795
	10-11	63.430	126.1	+ 612.7	+ 438.3	+ 0.7154
	12-13	56.800	126.1	+ 732.2	+ 457.1	+ 0.6244
	14-15	51.273	126.1	+ 993.7	+ 498.4	+ 0.5016
	16-17	46.850	126.1	+ 1 826.7	+ 629.8	+ 0.3448
	18-19	43.540	126.1	— 80 442.0	— 12 410.	+ 0.1543
	20-21	41.333	126.1	— 1 556.5	+ 94.6	— 0.0608
	22-23	40.220	126.1	— 769.7	+ 220.0	— 0.3776†

respectively. In the foregoing, x is the horizontal distance of C from A, and n is the horizontal distance of the panel point, D, from A, in units of the panel length, λ.

Similarly, Fig. 52 shows the elastic line, if the vertical, $F\ F'$, is considered elastic only, and the corresponding elastic loads at the panel points, F and G, are:

$$\Delta\ \alpha' = \Delta\ \alpha\left[\frac{x}{\lambda} - (n + 1)\right]$$

$$\Delta\ \alpha'' = \Delta\ \alpha\left(\frac{x}{\lambda} - n\right).$$

Tables 9 and 10 (Plate XXXII) show the calculation of the values, $\Delta\ \alpha'$ and $\Delta\ \alpha''$, again in units of 1 000 E. The resultant elastic panel loads from all truss members are given in Table 12. The moments due to these resultant elastic panel loads have been determined analytically

Δs, $\Delta \alpha$, AND Δl, FOR WEB MEMBERS.

(6)	(7)	(8)	(9)
$1\,000\,E\,\Delta s = 1\,000\,\frac{S_1 s}{A}$	$1\,000\,E\,\Delta \alpha = 1\,000\,\frac{E\,\Delta s}{r}$	$1\,000\,E\,\Delta l = \frac{y}{r}\,1\,000\,E\,\Delta s = y\,1\,000\,E\,\Delta \alpha$	Notes.
— 404.9	+ 1.8224	+ 345.6	* The stress, S_1, in the center diagonals has no influence on the deflection line.
— 359.4	+ 1.4551	+ 327.0	† The stress in 22–23 is corrected for effect of double diagonals in center panel.
— 334.0	+ 1.0379	+ 296.4	$\frac{y}{r}$ .... $= -0.2858$
— 446.8	+ 1.1126	+ 379.9	Correction ($H = 1$, $P = 0$)... $= -0.0918$
— 434.0	+ 1.1194	+ 382.2	Stress S_1...................... $= -0.3776$
— 420.4	+ 1.1062	+ 377.6	
— 401.1	+ 1.0470	+ 357.3	
— 284.3	+ 0.7016	+ 239.4	
— 244.4	+ 0.5250	+ 179.3	
— 159.2	+ 0.2585	+ 88.3	
— 88.7	+ 0.0797	+ 27.1	
	$1\,000\,E\,\frac{\Sigma\,\Delta\,l}{2} =$	+ 3 000.1	
$1\,000\,E\,\Sigma\,\Delta\,l$ for diagonals =		+ 6 000.2	
+ 387.1	+ 9.109	+ 334.0	
+ 427.6	+ 2.114	+ 412.9	
+ 396.0	+ 1.671	+ 396.5	
+ 482.0	+ 1.360	+ 438.5	
+ 404.9	+ 0.800	+ 315.6	Summary of values........ $1\,000\,E\,\Sigma\,\Delta\,l$
+ 359.9	+ 0.737	+ 257.5	
+ 281.2	+ 0.384	+ 175.6	Bottom chord .... = 18 143.8
+ 203.9	+ 0.205	+ 102.3	Top chord.................. = 36 127.8
+ 128.1	+ 0.0701	+ 44.2	Diagonals.................. = 6 000.2
+ 53.3	— 0.00066	+ 8.2	Verticals.................. = 5 041.8
— 19.9	+ 0.0128	+ 1.2	
— 120.4	+ 0.156	+ 34.4	Total..........$1\,000\,E\,\Sigma\,\Delta\,l =$ 65 313.6
	$1\,000\,E\,\frac{\Sigma\,\Delta\,l}{2} =$	+ 2 520.9	
$1\,000\,E\,\Sigma\,\Delta\,l$ for verticals =		+ 5 041.8	

in Table 12 by the usual method of shears and moment increments. The ordinate, δ, of the elastic line at any panel point, is equal to the corresponding moment from the elastic panel loads.

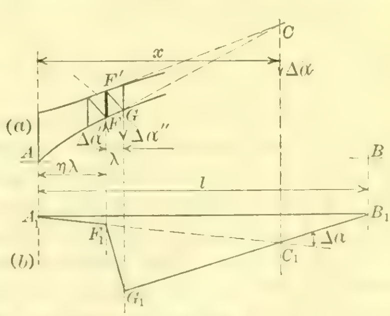


FIG. 52.

C.—Determination of Influence Line for Horizontal Reaction.—The influence ordinates for the horizontal reaction were determined in Table 12 by dividing the corresponding ordinates, δ, of the elastic line by the constant, $\Sigma\,\Delta\,l$ ($\Sigma\,\Delta\,l =$ horizontal deflection of the point, B, due to a horizontal load of unity at B, as obtained in Table 8, Column 9).

This is derived as follows: Assume the arch to be a simple span, $A B$, fixed at A and free to move at B (Fig. 53). According to Maxwell's Principle of Reciprocity, the vertical deflection, δ, at any point, C, due to a horizontal force of unity at B (Fig. 53 (a) and (b)), is equal to the horizontal deflection, ϵ, of the point, B, due to a vertical force of unity at C (Fig. 53 (c)), that is $\delta = \epsilon$.

In order to transform the simple span into a two-hinged arch, the horizontal reaction, H, has to overcome the horizontal deflection, ϵ, and we have, therefore, the relation, $\frac{H}{H \text{ unity}} = \frac{\varepsilon}{\Sigma \Delta l}$; or, as $\varepsilon = \delta$, $H = \frac{\delta}{\Sigma \Delta l}$.

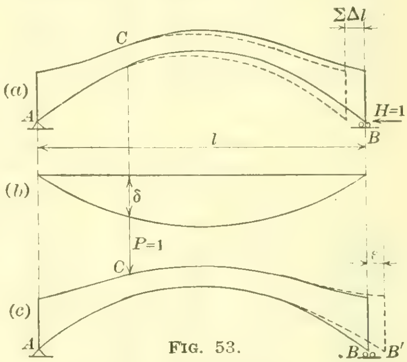


FIG. 53.

D.—Corrections for the Verticals, 0-1, 2-3, 4-5.—In the foregoing it has been assumed that the vertical load of unity is applied at the bottom chord panel points. As the live load is actually applied at the floor level, the following correction has to be made in the influence ordinates for H below the panel points, 0, 2, and 4.

Let s be the total length of the vertical and s' its length below the floor (Fig. 54). As the load is to be applied at C' instead of C, the numerator, δ, in the foregoing formula, $H = \frac{\delta}{\Sigma \Delta l}$, must be corrected to represent the deflection of C' instead of C. If δ is the deflection at C, $\delta + \Delta s'$ is the deflection at C'. The resulting change in the value of H is $\frac{\Delta s'}{\Sigma \Delta l}$. As $\Delta s' = \Delta s \cdot \frac{s'}{s}$, the correction for H is $\frac{\Delta s}{\Sigma \Delta l} \cdot \frac{s'}{s}$. (See Table 11.)

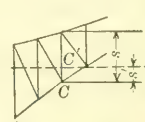


FIG. 54.

E.—Correction for Two Diagonals in the Center Panel.—The presence of two diagonals in the center panel adds another element of indeterminateness to the design. This is taken into account as follows: A load, P, is considered as acting at any point distant m panels from the end, A. The resulting stress in each center diagonal is that given by one-half the shear in the panel plus a correction, X, and the corresponding corrections in the other members of the center panel will be the horizontal and vertical components of X. The value which X must have in order to make both diagonals fit into their panel frame is then given by writing out and solving the equation, $\Delta d_1 + \Delta d_2 = (\Delta u + \Delta l)$ cos $B + (\Delta v_1 + \Delta v_2)$ sin. B, where Δu, Δl, Δv, and Δd

TABLE 9.—DETERMINATION OF ELASTIC PANEL LOADS, $\Delta\ \alpha' + \Delta\ \alpha''$, FOR DIAGONALS.

Panel length, $\lambda = 42.5$ ft.

Member.	(1) 1-2		(2) 3-4		(3) 5-6		(4) 7-8		(5) 9-10		(6) 11-12		(7) 13-14		(8) 15-16		(9) 17-18		(10) 19-20		(11) 21-22		(12) 23-22A	
$1000\ E\ \Delta\ a$ from Table 8........	+ 1.8224		+ 1.4551		+ 1.0379		+ 1.1126		+ 1.1194		+ 1.062		+ 1.0470		+ 0.7016		+ 0.5250		+ 0.2585		+ 0.0797			
x................	+ 219.83		+ 282.45		+ 390.45		+ 512.24		+ 561.12		+ 619.10		+ 691.75		+ 790.20		+ 941.55		+ 1 220.89		+ 2 003.03		∞	
n................	0		1		2		3		4		5		6		7		8		9		10		11	
Panel point.	0	2	2	4	4	6	6	8	[illegible]	10	10	12	12	14	14	16	16	18	18	20	20	22		
$1000\ E\ \Delta\ a' = 1000\ E\ \Delta\ a\left[\frac{x}{\lambda} - (n+1)\right]$..	− 7.603	....	− 6.756	....	− 6.480	....	− 8.962	....	− 9.182	....	− 9.477	....	− 9.712	....	− 7.432	....	− 6.906	....	− 4.840	....	− 2.879			
$1000\ E\ \Delta\ a'' = 1000\ E\ \Delta\ a\left[\frac{x}{\lambda} - n\right]$......	....	+ 9.425	....	+ 8.210	....	+ 7.460	....	+ 10.075	....	+ 10.302	....	+ 10.583	....	+ 10.759	....	+ 8.183	....	+ 7.431	....	+ 5.099	....	+ 2.959		
Total elastic panel load, $1000\ E\ [\Delta\ a' + \Delta\ a'']$......	− 7.603	+ 2.670		+ 1.788		− 1.502		+ 0.893		+ 0.825		+ 0.871		+ 3.327		+ 1.227		+ 2.591		+ 2.220		+ 2.959		

TABLE 10.—DETERMINATION OF ELASTIC PANEL LOADS, $\Delta\ \alpha' + \Delta\ \alpha''$, FOR VERTICALS.

Panel length, $\lambda = 42.5$ ft.

Member.	(1) 0-1		(2) 2-3		(3) 4-5		(4) 6-7		(5) 8-9		(6) 10-11		(7) 12-13		(8) 14-15		(9) 16-17		(10) 18-19		(11) 20-21		(12) 22-23	
$1000\ E\ \Delta\ a$ from Table 8.........	+ 9.1090		+ 2.1145		+ 1.6708		+ 1.3605		+ 0.7367		+ 0.5874		+ 0.3841		+ 0.2052		+ 0.0701		− 0.0007		+ 0.0123		+ 0.1565	
x................	+ 42.50		+ 244.74		+ 321.98		+ 481.77		+ 719.61		+ 825.18		+ 987.16		+ 1 291.22		+ 2 116.72		− 80 066.0		− 1 131.54		− 302.21	
n................	0		1		2		3		4		5		6		7		8		9		10		11	
Panel point.	0	2	2	4	4	6	6	8	8	10	10	12	12	14	14	16	16	18	18	20	20	22	22	22A
$1000\ E\ \Delta\ a' = 1000\ E\ \Delta\ a\left[\frac{x}{\lambda} - (n+1)\right]$..	0	... .	− 7.947	....	− 7.646	....	− 9.980	....	− 8.790	....	− 7.880	....	− 6.233	....	− 4.593	....	− 2.944	....	− 1.254	....	+ 0.482	....	+ 0.156	....
$1000\ E\ \Delta\ a'' = 1000\ E\ \Delta\ a\left[\frac{x}{\lambda} - n\right]$.....	....	+ 9.109	....	+ 10.062	....	+ 9.315	....	+ 11.341	....	+ 9.527	....	+ 8.467	....	+ 6.617	....	+ 4.798	....	+ 3.014	....	+ 1.255	....	− 0.469	....	....
Total elastic panel load, $1000\ E\ [\Delta\ a' + \Delta\ a'']$........	0	+ 1.162		+ 2.416		− 0.665		+ 2.551		+ 1.647		+ 2.234		+ 2.024		+ 1.854		+ 1.760		+ 1.737		− 0.313		

TABLE 11.—CORRECTION OF INFLUENCE ORDINATES FOR HORIZONTAL REACTION DUE TO LOADS APPLIED TO VERTICALS AT FLOOR LEVEL.

$1\,000\ E\ \Sigma\ \Delta\ l = 65\,313.6$.

Member.	(1) Total length, s, in feet.	(2) Length below floor, s' in feet.	(3) $1\,000\ E\,\Delta\,s = \frac{1\,000\ S\,s}{A}$. From Table 8.	(4) Ratio $\frac{A}{A'}$.	(5) Correction. $\frac{1\,000\ E\,\Delta\,s}{1\,000\ E\,\Sigma\,\Delta\,l} \times \frac{s'}{s} \times \frac{A}{A'}$
0–1	140.00	95	387.13	$\frac{312}{315} = 0.99$	0.00396
2–3	112.98	60	427.64	$\frac{255}{281} = 0.91$	0.00315
4–5	92.95	26	395.95	$\frac{235}{267} = 0.88$	0.00149

denote the elastic elongation of the top, bottom, vertical, and diagonal members, respectively, of the center panel, and is the inclination of the diagonals to the horizontal. We thus obtain

$$X = 0.1335\ H - 0.01538\ m\ P.$$

Consequently, for the sole application of $H = 1$, $X = +\ 0.1335$ is the stress in each center diagonal, and the resulting corrections in the other members of the center panel are included in Tables 7 and 8. The corrected value of the influence line for H is thus found, although the effect on H of the foregoing correction proves to be quite negligible.

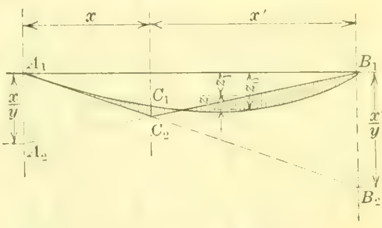


FIG. 55.

Substituting the resulting values of H with the corresponding values of m in the foregoing equation, we obtain the influence values of X for a unit load at the successive panel points of the span. These values are tabulated as the influence ordinates for the center diagonal in Table 15 (Plate XXXIV), and the corresponding corrections are tabulated for the other members of the center panel in Tables 13 and 14 (Plate XXXIII), and Table 16 (Plate XXXIV). All the remaining members of the truss are unaffected by the double center diagonals.

2.—Influence Lines for Arch Truss Members.

The influence ordinates for the stresses in the members of the arch truss were determined analytically as follows:

Assuming the truss to be a simple span, the influence line for the stress in any member has two straight segments, $A_1\ B_2$ and $B_1\ A_2$ (Fig. 55), which intersect at C_2 vertically below the center of moments, C_1 of the member, and have the ordinates, A_1, $A_2 = \frac{x}{r}$, and B_1, $B_2 = \frac{x'}{r}$.

TABLE 12.—DETERMINATION OF INFLUENCE ORDINATES FOR HORIZONTAL REACTION.

Panel Length, λ = 42.5 ft. 1 000 $E \Sigma \Delta l$ = 65 313.6.

	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)
Panel point.	0	2	4	6	8	10	12	14	16	18	20	22	Sum of elastic panel loads for ½ truss.
Elastic panel loads. 1 000 $E \Delta a$. Bottom chord. From Table 7	....	+ 0.799	+ 1.163	+ 1.471	+ 2.19	+ 2.715	+ 3.428	+ 4.229	+ 5.246	+ 6.233	+ 6.971	+ 3.735	
Elastic panel loads. 1 000 $E \Delta a$. Top chord. From Table 7.	....	+ 0.418	+ 1.259	+ 2.638	+ 4.121	+ 4.976	+ 6.702	+ 8.659	+ 10.731	+ 12.686	+ 14.230	{ + 17.645 / + 8.627	
Elastic panel loads. 1 000 $E (\Delta a' + \Delta a'')$. Diagonals. From Table 9.	....	+ 2.670	+ 1.788	− 1.502	+ 0.893	+ 0.825	+ 0.871	+ 3.327	+ 1.227	+ 2.591	+ 2.220	+ 2.959	
Elastic panel loads. 1 000 $E (\Delta a' + \Delta a'')$. Verticals. From Table 9.	....	+ 1.162	+ 2.416	− 0.665	+ 2.551	+ 1.647	+ 2.234	+ 2.024	+ 1.854	+ 1.760	+ 1.737	− 0.313	
Total elastic panel load...	....	+ 5.049	+ 6.626	+ 0.942	+ 9.684	+ 10.163	+ 13.235	+ 18.239	+ 19.058	+ 23.270	+ 25.158	+ 32.653	+ 165.077
Shear = V. Moment increment = $V\lambda$	165.077 7 015.9	160.028 6 801.3	153.402 6 519.5	151.460 6 437.0	141.776 6 025.5	131.613 5 593.4	118.378 5 031.1	100.139 4 255.9	81.081 3 445.9	57.811 2 457.0	32.653 1 387.8		
Moment = ordinate of elastic line = 1 000 $E\delta$	0	7 015.9	13 817.2	20 336.7	26 773.7	32 799.2	38 392.6	43 423.7	47 679.6	51 125.5	53 582.5	54 970.3	
Influence ordinates for horizontal reaction = $\frac{1\,000\,E\delta}{1\,000\,E\Sigma\Delta l}$	0	0.1074	0.2115	0.311	0.410	0.502	0.588	0.665	0.730	0.783	0.820	0.841	
Correction for loads at floor level. From Table 11.	0.0040	0.0032	0.0015	....	....	....	....	....	....	....	....	....	Sum of ordinates for ½ truss.
Total influence ordinates for horizontal reaction.	0.004	0.111	0.213	0.311	0.410	0.502	0.588	0.665	0.730	0.783	0.820	0.841	5.978

NOTE.—1 000 $E \Sigma \Delta l$ = 65 313.6, therefore, $\frac{65\,313.6 \times 12}{30\,000} = 26.125$ in., shortening for H = 1 000 000 lb. For 1 in. shortening, $H = \frac{1\,000\,000}{26.125} = 38\,278$ lb.

TABLE 13.—Determination of Influence Ordinates.—For Bottom Chord. Live Load per Panel = 510 000 lb. = 1 000 P. All Stresses Given in Units of 1 000 lb.

+ Denotes tension.
— Denotes compression.

Panel Points.—All Values in These Columns are Influence Ordinates, and are Expressed in Thousandths of a Unit.

Member.	(1) X, in feet.	(2) X', in feet.	(3) $\frac{x}{y}$	(4) $\frac{x'}{y}$	(5) $\frac{y}{r}$	(6)	0	2	4	6	8	10	12	14	16	18	20	22
Influence ordinate. Hor. reaction	.....	.....	.....	.....	.....	Z_0	+4	+111	+213	+311	+410	+502	+588	+665	+730	+783	+820	+841
						Z_1	0											
0-2	0	977.5	0.000	-6.982	-1.321	$Z = Z_0 + Z_1$	+4	+111	+213	+311	+410	+502	+588	+665	+730	+783	+820	+841
						Z_1	0	-272	-259	-247	-235	-222	-210	-198	-185	-173	-161	-148
2-4	42.5	935.0	-0.284	-6.250	-1.684	$Z = Z_0 + Z_1$	+4	-161	-46	+64	+175	+280	+378	+467	+545	+610	+659	+693
						Z_1	0	-238	-476	-454	-431	-408	-386	-363	-340	-318	-295	-272
4-6	85.0	892.5	-0.522	-5.478	-2.146	$Z = Z_0 + Z_1$	+4	-127	-263	-143	-21	+94	+202	+302	+390	+465	+525	+569
						Z_1	0	-205	-411	-616	-585	-554	-524	-493	-462	-431	-400	-370
6-8	127.5	850.0	-0.708	-4.722	-2.656	$Z = Z_0 + Z_1$	+4	-94	-198	-305	-175	-52	+64	+172	+268	+352	+420	+471
						Z_1	0	-177	-355	-532	-710	-672	-635	-598	-560	-523	-486	-448
8-10	170.0	807.5	-0.859	-4.032	-3.171	$Z = Z_0 + Z_1$	+4	-66	-142	-221	-300	-170	-47	+67	+170	+260	+334	+393
						Z_1	0	-156	-312	-467	-623	-779	-736	-693	-649	-606	-563	-519
10-12	212.5	765.0	-0.996	-3.584	-3.719	$Z_1 = Z_0 + Z_1$	+4	-45	-99	-156	-213	-277	-148	-28	+81	+177	+257	+322
						Z_1	0	-139	-277	-416	-554	-692	-831	-782	-733	-684	-635	-587
12-14	255.0	722.5	-1.124	-3.186	-4.289	$Z = Z_0 + Z_1$	+4	-28	-64	-105	-144	-190	-243	-117	-3	+99	+185	+254
						Z_1	0	-124	-249	-373	-497	-621	-746	-870	-815	-761	-707	-652
14-16	297.5	680.0	-1.250	-2.858	-4.864	$Z = Z_0 + Z_1$	+4	-13	-36	-62	-87	-119	-158	-205	-85	+22	+113	+189
						Z_1	0	-112	-225	-337	-449	-561	-674	-786	-898	-838	-779	-719
16-18	340.0	637.5	-1.377	-2.582	-5.413	$Z = Z_0 + Z_1$	+4	-1	-12	-26	-39	-59	-86	-121	-168	-55	+41	+122
						Z_1	0	-102	-204	-306	-408	-510	-612	-714	-816	-918	-853	-787
18-20	382.5	595.0	-1.509	-2.347	-5.895	$Z = Z_0 + Z_1$	+4	+9	+9	+5	+2	-8	-24	-49	-86	-135	-33	+54
						Z_1	0	-93	-186	-279	-372	-466	-559	-652	-745	-838	-931	-859
20-22	425.0	552.5	-1.647	-2.142	-6.261	$Z = Z_0 + Z_1$	+4	+18	+27	+32	+38	+36	+29	+13	-15	-55	-111	-18
						Z_1	0	-82	-163	-245	-327	-408	-490	-572	-653	-735	-817	-898
22-22A	488.75	488.75	-1.878	-1.878	-6.470	Correct Z_2	+0.06	-0.07	-0.27	-0.52	-0.75	-1.10	-1.55	-2.12	-2.86	-3.81	-4.97	-6.37
						$Z = Z_0 + Z_1 + Z_2$	+5	+29	+50	+65	+82	+93	+96	+91	+74	+44	-2	-63

Member.	22	20	18	16	14	12	10	8	6	4	2	0	(31) Influence Area, $Z = \Sigma z$. Positive. +	(32) Negative. —	(33) Total.	(34) Live-Load Stress $= P \frac{y}{r} \Sigma z$. +	(35) —
Influence ordinate. Hor. reaction	+841	+820	+783	+730	+665	+588	+502	+410	+311	+213	+111	+4	.....	.....	.....	.....	.....
												0					
0-2	+841	+820	+783	+730	+665	+588	+502	+410	+311	+213	+111	+4	11.962	0.	+11.962	.....	8 057
	-136	-124	-111	-99	-86	-74	-62	-49	-37	-25	-12	0					
2-4	+705	+696	+672	+631	+579	+514	+440	+361	+274	+188	+99	+4	9.021	0.192	+8.829	165	7 746
	-250	-227	-204	-182	-159	-136	-113	-91	-68	-45	-23	0					
4-6	+591	+593	+579	+548	+506	+452	+389	+319	+243	+168	+88	+4	7.019	0.543	+6.476	524	7 682
	-339	-308	-277	-246	-216	-185	-154	-123	-92	-62	-31	0					
6-8	+502	+512	+506	+484	+449	+403	+348	+287	+219	+151	+80	+4	5.678	0.808	+4.870	1 095	7 692
	-411	-374	-336	-299	-262	-224	-187	-149	-112	-75	-37	0					
8-10	+430	+446	+447	+431	+403	+364	+315	+261	+199	+138	+74	+4	4.722	0.930	+3.792	1 504	7 638
	-476	-433	-390	-346	-303	-260	-216	-173	-129	-86	-43	0					
10-12	+365	+387	+393	+384	+362	+328	+286	+237	+182	+127	+68	+4	3.950	0.954	+2.996	1 809	7 492
	-538	-489	-440	-391	-342	-293	-244	-196	-147	-98	-49	0					
12-14	+303	+331	+343	+339	+323	+295	+258	+214	+164	+115	+62	+4	3.288	0.891	+2.397	1 949	7 192
	-598	-544	-489	-435	-381	-326	-272	-217	-163	-109	-54	0					
14-16	+243	+276	+294	+295	+284	+262	+230	+193	+148	+104	+57	+4	2.706	0.755	+1.951	1 873	6 712
	-659	-599	-539	-479	-419	-359	-300	-240	-180	-120	-60	0					
16-18	+182	+221	+244	+251	+246	+229	+203	+170	+131	+93	+51	+4	2.178	0.555	+1.623	1 532	6 013
	-722	-656	-590	-525	-459	-394	-328	-262	-197	-131	-66	0					
18-20	+119	+164	+193	+205	+206	+194	+174	+148	+114	+82	+45	+4	1.718	0.324	+1.394	974	5 165
	-788	-716	-645	-573	-501	-430	-358	-287	-215	-143	-72	0					
20-22	+53	+104	+138	+157	+164	+158	+144	+123	+96	+70	+39	+4	1.438	0.188	+1.250	600	4 591
	-898	-817	-735	-653	-572	-490	-408	-327	-245	-163	-82	0					
22-22A	-6.37	-4.97	-3.81	-0.86	-2.12	-1.55	-1.10	-0.75	-0.52	-0.27	-0.07	+0.06	1.252	0.128	+1.124	422	4 131
	-63	-2	+44	+74	+91	+96	+93	+82	+65	+50	+29	+5					

Stress 22-22A $= -0.47\,\eta_{II} + 0.5288\,m - \eta_{x}' \cdot \cos\beta$
$= -0.47\,(\eta_{II} - 0.09105\,m + 0.1193\,\eta_{x}')$
$= -0.47\,(Z_0 + Z_1 + Z_2)$.

TABLE 14.—Determination of Influence Ordinates.—For Top Chord. Live Load per Panel = 510 000 lb. = 1 000 P. All Stresses Given in Units of 1 000 lb.

+ Denotes tension.
— Denotes compression.

Panel Points.—All Values in These Columns Are Influence Ordinates, and Are Expressed in Thousandths of a Unit.

Member.	(1) X, in feet.	(2) X', in feet.	(3) $\frac{x}{y}$	(4) $\frac{x'}{y}$	(5) $\frac{y}{r}$	(6)	0	2	4	6	8	10	12	14	16	18	20	22
Influence ordinate. Hor. reaction	.....	.....	.....	.....	.....	Z_0	+4	+111	+213	+311	+410	+502	+588	+665	+730	+783	+820	+841
						Z_1	0	-1 109	-1 059	-1 008	-958	-907	-857	-807	-756	-706	-655	-605
1-3	42.5	935.0	-1.159	-25.500	+0.833	$Z = Z_0 + Z_1$	+4	-998	-846	-697	-548	-405	-269	-142	-26	+77	+165	+236
						Z_1	0	-554	-1 109	-1 056	-1 003	-950	-898	-845	-792	-739	-686	-634
3-5	85.0	892.5	-1.214	-12.750	+0.789	$Z = Z_0 + Z_1$	+4	-443	-896	-745	-593	-448	-310	-180	-62	+44	+134	+207
						Z_1	0	-370	-739	-1 109	-1 053	-998	-942	-887	-832	-776	-721	-665
5-7	127.5	850.0	-1.275	-8.500	+1.347	$Z = Z_0 + Z_1$	+4	-259	-526	-798	-643	-496	-354	-222	-102	+7	+99	+176
						Z_1	0	-277	-554	-832	-1 109	-1 050	-992	-934	-875	-816	-759	-700
7-9	170.0	807.5	-1.342	-6.375	+1.980	$Z = Z_0 + Z_1$	+4	-166	-341	-521	-699	-548	-404	-269	-145	-34	+61	+141
						Z_1	0	-222	-443	-665	-887	-1 109	-1 047	-986	-924	-862	-801	-739
9-11	212.5	765.0	-1.417	-5.100	+2.519	$Z = Z_0 + Z_1$	+4	-111	-230	-354	-477	-607	-459	-321	-194	-79	+19	+102
						Z_1	0	-185	-370	-554	-739	-924	-1 109	-1 043	-978	-913	-848	-783
11-13	255.0	722.5	-1.500	-4.25	+3.188	$Z = Z_0 + Z_1$	+4	-74	-157	-243	-329	-422	-521	-378	-248	-130	-28	+58
						Z_1	0	-158	-317	-476	-634	-792	-950	-1 109	-1 039	-970	-901	-831
13-15	297.5	680.0	-1.593	-3.648	+3.768	$Z = Z_0 + Z_1$	+4	-47	-104	-165	-224	-290	-362	-444	-309	-187	-81	+10
						Z_1	0	-139	-277	-416	-554	-693	-832	-970	-1 109	-1 035	-961	-887
15-17	340.0	637.5	-1.700	-3.188	+4.365	$Z = Z_0 + Z_1$	+4	-28	-64	-105	-144	-191	-244	-305	-379	-252	-141	-46
						Z_1	0	-123	-246	-370	-493	-616	-739	-862	-986	-1 109	-1 030	-950
17-19	382.5	595.0	-1.821	-2.833	+4.883	$Z = Z_0 + Z_1$	+4	-12	-33	-59	-83	-114	-151	-197	-256	-326	-210	-109
						Z_1	0	-111	-222	-333	-444	-554	-665	-776	-887	-998	-1 109	-1 024
19-21	425.0	552.5	-1.962	-2.550	+5.270	$Z = Z_0 + Z_1$	+4	0	-9	-22	-34	-52	-77	-111	-157	-215	-289	-183
						Z_1	0	-101	-202	-302	-403	-504	-605	-706	-806	-907	-1 008	-1 109
21-23	467.5	510.0	-2.125	-2.319	+5.478	$Z = Z_0 + Z_1$	+4	+10	+11	+9	+7	-2	-17	-41	-76	-124	-188	-268
						Z_1	0	-96.6	-193.2	-289.8	-386.4	-483	-579.5	-676.1	-772.7	-869.3	-965.9	-1 062.5
23-23A	488.75	488.75	-2.222	-2.222	+5.470	Correct Z_2	-0.07	+0.1	+0.3	+0.6	+0.9	+1.3	+1.8	+2.5	+3.4	+4.5	+5.9	+7.5
						$Z = Z_0 + Z_1 + Z_2$	+4	+14	+20	+22	+24	+20	+10	-9	-39	-82	-140	-214

Member.	22	20	18	16	14	12	10	8	6	4	2	0	(31) Influence Area, $Z = \Sigma z$. Positive. +	(32) Negative. —	(33) Total.	(34) Live-Load Stress $= P \frac{y}{r} \Sigma z$. +	(35) —
Influence ordinate. Hor. reaction	+841	+820	+783	+730	+665	+588	+502	+410	+311	+213	+111	+4	.....	.....	.....	.....	.....
	-555	-504	-454	-403	-353	-302	-252	-202	-151	-101	-50	0					
1-3	+286	+316	+329	+327	+312	+286	+250	+208	+160	+112	+61	+4	3.119	3.916	-0.797	530	665
	-581	-528	-475	-422	-370	-317	-264	-211	-153	-106	-53	0					
3-5	+260	+293	+308	+308	+295	+271	+238	+199	+158	+107	+58	+4	2.865	3.662	-0.797	1 153	1 474
	-610	-554	-499	-444	-388	-333	-277	-222	-166	-111	-55	0					
5-7	+231	+266	+284	+286	+277	+255	+225	+188	+145	+102	+56	+4	2.598	3.395	-0.797	1 785	2 332
	-642	-584	-525	-467	-408	-350	-292	-234	-175	-117	-58	0					
7-9	+199	+236	+258	+263	+257	+238	+210	+176	+136	+96	+53	+4	2.317	3.114	-0.797	2 281	3 006
	-678	-616	-554	-493	-431	-370	-308	-246	-185	-123	-62	0					
9-11	+163	+204	+229	+237	+234	+218	+194	+164	+126	+90	+49	+4	2.025	2.822	-0.797	2 602	3 626
	-717	-652	-587	-522	-456	-391	-326	-261	-196	-130	-65	0					
11-13	+124	+168	+196	+208	+209	+197	+176	+149	+115	+83	+46	+4	1.724	2.519	-0.797	2 759	4 031
	-762	-693	-624	-554	-485	-416	-347	-277	-208	-139	-69	0					
13-15	+79	+127	+159	+176	+180	+172	+155	+133	+103	+74	+42	+4	1.410	2.207	-0.797	2 706	4 236
	-813	-739	-665	-591	-517	-443	-370	-296	-222	-148	-74	0					
15-17	+28	+81	+118	+139	+148	+145	+132	+114	+89	+65	+37	+4	1.092	1.889	-0.797	2 429	4 202
	-871	-792	-713	-634	-554	-475	-396	-317	-238	-158	-79	0					
17-19	-30	+28	+70	+96	+111	+113	+106	+93	+73	+55	+32	+4	0.774	1.571	-0.797	1 927	3 912
	-938	-853	-768	-683	-597	-512	-427	-341	-256	-171	-85	0					
19-21	-97	-33	+15	+47	+68	+76	+75	+69	+55	+42	+26	+4	0.475	1.272	-0.797	1 277	3 419
	-1 016	-924	-832	-739	-647	-554	-462	-370	-277	-185	-92	0					
21-23	-175	-104	-49	-9	+18	+34	+40	+40	+34	+28	+19	+4	0.252	1.049	-0.797	704	2 931
	-1 062.5	-965.9	-869.3	-772.7	-676.1	-579.5	-483	-386.4	-289.8	-193.2	-96.6	0					
23-23A	+7.5	+5.9	+4.5	+3.4	+2.5	+1.8	+1.3	+0.9	+0.6	+0.3	+0.1	-0.07	0.221	0.963	-0.742	617	2 686
	-214	-140	-82	-39	-9	+10	+20	+24	+22	+20	+14	+4					

Stress 23-23A $= +5.47\,(\eta_{II} - 0.0966\,m - 0.1328\,\eta_{x}')$
$= +5.47\,(Z_0 + Z_1 + Z_2)$.

TABLE 15.—DETERMINATION OF INFLUENCE ORDINATES.—FOR DIAGONALS. LIVE LOAD PER PANEL = 510 000 LB. = 1 000 P. ALL STRESSES GIVEN IN UNITS OF 1 000 LB.

+ Denotes tension.
— Denotes compression.

PANEL POINTS.—ALL VALUES IN THESE COLUMNS ARE INFLUENCE ORDINATES, AND ARE EXPRESSED IN THOUSANDTHS OF A UNIT.

Member	(1) X, in feet.	(2) X', in feet.	(3) $\frac{x}{y}$	(4) $\frac{x'}{y}$	(5) $\frac{y}{r}$	(6)	0	2	4	6		10	12	14	16	18	20	22
Influence ordinate. Hor. reaction	......	......	......	......	......	Z_0	+4	+111	+218	+311	+410	+502	+588	+665	+730	+783	+820	+841
1-2	219.83	757.67	1.159	-8.995	-0.854	Z_1	0	-1 109	-1 058	-1 008	-958	-907	-856	-806	-756	-706	-655	-605
						$Z = Z_0 + Z_1$	+4	-998	-845	-697	-548	-405	-268	-141	-26	+77	+165	+236
3-4	282.45	695.05	-1.256	-8.089	-0.910	Z_1	0	-134	-1 147	-1 092	-1 038	-983	-929	-874	-819	-765	-710	-655
						$Z = Z_0 + Z_1$	+4	-23	-934	-781	-628	-481	-341	-209	-89	+18	+110	+186
5-6	390.45	587.05	-1.367	-2.056	-0.888	Z_1	0	89	-179	-1 189	-1 129	-1 070	-1 011	-951	-892	-832	-773	-713
						$Z = Z_0 + Z_1$	+4	22	+84	-878	-719	-568	-423	-286	-162	-49	+47	+128
7-8	512.84	465.25	1.600	-1.368	0.850	Z_1	0	59	118	-178	-1 239	-1 174	-1 109	-1 044	-979	-913	-848	-783
						$Z = Z_0 + Z_1$	+4	52	95	-133	-829	-672	-521	-379	-249	-130	-28	+58
9-10	561.12	416.38	-1.648	-1.220	0.890	Z_1	0	53	106	159	-212	-1 286	-1 215	-1 144	-1 072	-1 001	-929	-857
						$Z = Z_0 + Z_1$	+4	58	+107	+152	+198	-784	-627	-479	-342	-218	-109	-16
11-12	619.10	358.40	1.814	1.050	0.898	Z_1	0	46	91	137	-183	-228	-1 341	-1 262	-1 183	-1 104	-1 025	-946
						$Z = Z_0 + Z_1$	+4	65	+122	+174	+227	+274	-753	-597	-453	-321	-205	-105
13-14	691.75	285.75	2.027	0.887	0.891	Z_1	0	36	73	109	146	182	-218	-1 410	-1 322	-1 234	-1 146	-1 058
						$Z = Z_0 + Z_1$	+4	75	+140	+202	+264	+320	+370	-745	-592	-451	-326	-217
15-16	790.20	187.30	2.816	-0.549	-0.842	Z_1	0	24	48	72	95	119	143	167	-1 510	-1 410	-1 309	-1 208
						$Z = Z_0 + Z_1$	+4	87	165	+239	+315	+383	+445	+498	-780	-627	-489	-367
17-18	941.55	85.95	2.757	0.105	0.734	Z_1	0	5	9	14	18	23	27	32	37	-1 678	-1 558	-1 438
						$Z = Z_0 + Z_1$	+4	106	+204	+297	+392	+479	+561	+633	+693	-895	-738	-597
19-20	1 220.69	248.89	3.575	+0.718	0.554	Z_1	0	31	62	93	124	155	186	217	248	+279	-2 021	-1 805
						$Z = Z_0 + Z_1$	+4	142	+275	+404	+534	+657	+774	+882	+978	+1 062	-1 201	-1 024
21-22	2 003.46	1 025.58	5.894	3.012	0.306	Z_1	0	131	262	393	524	655	786	917	1 048	+1 179	+1 310	-3 070
						$Z = Z_0 + Z_1$	+4	244	475	+704	+934	+1 157	+1 374	+1 582	+1 778	+1 962	+2 130	-2 229
23-22	∞	......	......	......	......	Z_1	0	31.63	+63.25	+94.88	+126.51	+158.14	+189.76	+221.39	+253.02	+284.64	+316.27	+347.90
						Z_2	0.?	0.6	+2.4	+4.6	+6.7	+9.8	+13.8	+18.9	+25.5	+33.9	+44.3	+56.7
						$Z = Z_1 + Z_2$	0.5	32	+66	+99	+133	+168	+204	+240	+279	+319	+361	+405

Member	(6)	22	20	18	16	14	12	10	8	6	4	2	0	(31) Influence Area $Z = \Sigma z$. Positive. +	(32) Negative. —	(33) Total.	(34) Live-Load Stress $= P\frac{y}{r}\Sigma z$. +	(35) —
Influence ordinate. Hor. reaction	Z_0	+841	+820	+783	+730	+665	+588	+502	+410	+311	+218	+111	+4	......	......	......	......	......
1-2	Z_1	-554	-504	-455	-403	-353	-302	-252	-202	-151	-101	-50	0					
	$Z = Z_0 + Z_1$	+287	+316	+328	+327	+312	+286	+250	+208	+160	+112	+61	+4	3.119	3.916	-0.797	1 705	1 358
3-4	Z_1	-601	-546	-492	-437	-382	-328	-273	-218	-164	-109	-55	0					
	$Z = Z_0 + Z_1$	+240	+274	+291	+293	+283	+260	+229	+192	+147	+104	+56	+4	2.680	3.477	-0.797	1 613	1 243
5-6	Z_1	-654	-594	-535	-476	-416	-357	-297	-238	-178	-119	-59	0					
	$Z = Z_0 + Z_1$	+187	+226	+248	+254	+240	+231	+205	+172	+133	+94	+52	+4	2.200	3.057	0.797	1 334	1 023
7-8	Z_1	-718	-652	-587	-522	-457	-392	-327	-261	-196	-130	-65	0					
	$Z = Z_0 + Z_1$	+123	+168	+196	+209	+208	+196	+175	+149	+115	+83	+46	+4	1.944	2.741	-0.797	1 189	848
9-10	Z_1	-786	-715	-643	-572	-500	-429	-357	-286	-215	-148	-72	0					
	$Z = Z_0 + Z_1$	+55	+105	+140	+158	+165	+159	+145	+124	+96	+70	+39	+4	1.692	2.489	0.797	1 118	760
11-12	Z_1	-867	-789	-710	-631	-552	-473	-394	-315	-237	-158	-79	0					
	$Z = Z_0 + Z_1$	-26	+31	+73	+99	+113	+115	+108	+95	+74	+55	+32	+4	1.555	2.352	0.797	1 077	712
13-14	Z_1	-969	-881	-793	-705	-617	-529	-441	-353	-265	-176	-88	0					
	$Z = Z_0 + Z_1$	-128	-61	-10	+25	+48	+59	+61	+57	+46	+37	+23	+4	1.606	2.403	0.797	1 092	730
15-16	Z_1	-1 108	-1 007	-907	-806	-705	-604	-503	-403	-302	-201	-101	0					
	$Z = Z_0 + Z_1$	-267	-187	-124	-76	-40	-16	-1	+7	+9	+12	+10	+4	2.024	2.821	0.797	1 212	870
17-18	Z_1	-1 318	-1 199	-1 079	-959	-839	-719	-599	-480	-360	-240	-120	0					
	$Z = Z_0 + Z_1$	-477	-379	-296	-229	-174	-131	-97	-70	-49	-27	-9	+4	3.174	3.971	-0.797	1 485	1 187
19-20	Z_1	-1 710	-1 554	-1 399	-1 244	-1 088	-933	-777	-622	-467	-311	-155	0					
	$Z = Z_0 + Z_1$	-869	-734	-616	-514	-423	-345	-275	-212	-156	-98	-44	+4	5.431	6.228	-0.797	1 702	1 536
21-22	Z_1	-2 814	-2 559	-2 303	-2 047	-1 791	-1 535	-1 279	-1 023	-768	-512	-256	0					
	$Z = Z_0 + Z_1$	-1 973	-1 739	-1 520	-1 317	-1 126	-947	-777	-613	-457	-299	-145	+4	11.799	12.596	0.797	1 059	1 835
23-22	Z_1	-347.90	-316.27	-284.64	-253.02	-221.39	-189.76	-158.14	-126.51	-94.88	-63.25	-31.63	0					
	Z_2	+56.7	+44.3	+33.9	+25.5	+18.9	+13.8	+9.8	+6.7	+4.6	+2.4	+0.6	-0.5					
	$Z = Z_1 + Z_2$	-291	-272	-251	-228	-202	-176	-148	-120	-90	-61	-31	-0.5	2.221	1.786	0.435	911	1 133

Stress 23-22 $= \mp 0.08168\, m + \eta_z = (-1)[\pm 0.08168\, m - \eta_z] = (-1)[Z_0 + Z_1 + Z_2]$.

TABLE 16.—DETERMINATION OF INFLUENCE ORDINATES.—FOR VERTICALS. LIVE LOAD PER PANEL = 510 000 LB. = 1 000 P. ALL STRESSES GIVEN IN UNITS OF 1 000 LB.

+ Denotes tension.
— Denotes compression.

PANEL POINTS.—ALL VALUES IN THESE COLUMNS ARE INFLUENCE ORDINATES AND ARE EXPRESSED IN THOUSANDTHS OF A UNIT

Member.	(1) X, in feet	(2) X', in feet.	(3) $\frac{x}{y}$	(4) $\frac{x'}{y}$	(5) $\frac{y}{r}$	(6)	0	2	4	6	8	10	12	14	16	18	20	22
Influence ordinate. Hor. reaction	......	......	......	......		Z_0	+4	+111	+218	+311	+410	+502	+588	+665	+730	+783	+820	+841
0-1	42.50	935.00	1.159	25.600	0.863	Z_1	0	-1 109	-1 058	-1 008	-958	-907	-857	-806	-756	-706	-655	-605
					Above floor	$Z = Z_0 + Z_1$	+4	-998	-845	-697	-548	-405	-269	-141	-26	+77	+165	+236
					Below floor	$Z = Z_0 + Z_1$	1 165											
2-3	244.74	739.76	-1.258	3.752	+0.966	Z_1	0	163	-1 144	-1 090	-1 035	-981	-926	-872	-817	-763	-708	-654
						$Z = Z_0 + Z_1$		52										
						$Z = Z_0 + Z_1$	+4	-1 088	-931	-779	-625	-479	-338	-207	-87	+20	+112	+187
4-5	381.98	655.52	1.857	2.703	+1.001	Z_1	0	-120	-240	-1 180	-1 121	-1 062	-1 003	-944	-885	-826	-767	-709
						$Z = Z_0 + Z_1$			27									
						$Z = Z_0 + Z_1$	+4	-9	-1 026	-869	-711	-560	-415	-279	-155	-43	+58	+132
6-7	481.77	496.73	1.496	1.538	+0.910	Z_1	0	67	133	-201	-1 235	-1 170	-1 105	-1 040	-975	-910	-845	-780
						$Z = Z_0 + Z_1$	+4	44	+80	+110	-825	-668	-517	-375	-245	-127	-25	+61
8-9	719.61	257.89	1.690	0.602	+0.780	Z_1	0	26	52	79	-105	-1 315	-1 242	-1 169	-1 096	-1 022	-949	-877
						$Z = Z_0 + Z_1$	+4	85	+161	+232	+305	-818	-654	-504	-366	-239	-129	-36
10-11	825.18	152.32	-1.883	0.348	+0.715	Z_1	0	15	30	45	-60	-76	-1 392	-1 310	-1 228	-1 146	-1 064	-982
						$Z = Z_0 + Z_1$	+4	96	+183	+266	+350	+426	-804	-645	-498	-363	-214	-141
12-13	937.16	9.06	-2.160	0.021	+0.634	Z_1	0	1	2	3	4	5	6	-1 502	-1 408	-1 314	-1 221	-1 127
						$Z = Z_0 + Z_1$	+4	112	+215	+314	+414	+507	+594	-837	-678	-531	-401	-286
14-15	1 291.92	818.72	2.691	0.629	+0.509	Z_1	0	27	55	82	109	137	164	192	-1 690	-1 577	-1 464	-1 352
						$Z = Z_0 + Z_1$	+4	138	+273	+393	+519	+639	+752	+857	-960	-794	-644	-511
16-17	2 166.72	1 189.22	3.440	1.888	+0.345	Z_1	0	82	164	246	328	410	493	575	657	-2 094	-1 944	-1 795
						$Z = Z_0 + Z_1$	+4	193	+377	+557	+738	+912	+1 081	+1 240	+1 387	-1 311	-1 124	-954
18-19	80 066	81 019	-5.452	+6.530	+0.154	Z_1	0	284	568	852	+1 136	+1 420	-1 704	+1 988	+2 271	+2 555	-3 647	-3 366
						$Z = Z_0 + Z_1$	+4	395	+781	+1 163	+1 546	+1 922	-2 292	+2 653	+3 001	+3 338	-2 827	-2 525
20-21	1 131.54	2 109.01	+11.964	22.298	0.001	Z_1	0	969	1 934	2 909	-3 878	4 848	-5 817	-6 787	-7 756	-8 725	-9 695	+6 242
						$Z = Z_0 + Z_1$	+4	858	1 726	2 598	-3 468	4 346	-5 229	-6 122	-7 026	-7 942	8 875	+7 083
22-23	302.21	1 279.71	1.871	5.817	0.286	Z_1	0	177	354	531	707	884	-1 061	-1 238	1 415	1 592	1 768	-1 945
						Z_2	1.2	1.4	5.8	11.1	16.1	23.6	33.2	45.4	61.3	81.5	106.7	136.4
						$Z = Z_0 + Z_1 + Z_2$	+5	67	-147	-231	-318	-406	-506	-618	-746	-891	-1 055	-1 240

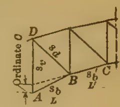
Member.	(6)	22	20	18	16	14	12	10	8	6	4	2	0	(31) Influence Area, $Z = \Sigma z$. Positive. +	(32) Negative. —	(33) Total.	(34) Live-Load Stress $= P\frac{y}{r}\Sigma z$. +	(35) —	
Influence ordinate. Hor. reaction	Z_0	+841	+820	+783	+730	+665	+588	+502	+410	+311	+218	+111	+4	......	......	......	......	......	
0-1	Z_1	-554	-504	-454	-403	-353	-302	-252	-202	-151	-101	-50	0	3.119	3.916	-0.797	1 872	1 722	Above floor
	$Z = Z_0 + Z_1$	+287	+316	+329	+327	+312	+286	+250	+208	+160	+112	+61	+4	3.119	4.783	-1.664	1 872	2 105	Below floor.
2-3	Z_1	-599	-545	-490	-436	-382	-327	-273	-218	-164	-109	-54	0	2.691	3.488	-0.797	1 325	1 718	Above floor.
	$Z = Z_0 + Z_1$	+242	+275	+293	+294	+283	+261	+229	+192	+147	+104	+57	+4	2.691	4.524	-1.833	1 325	2 228	Below floor.
4-5	Z_1	-649	-590	-531	-472	-413	-354	-295	-236	-177	-118	-59	0	2.258	3.055	-0.797	1 158	1 560	Above floor.
	$Z = Z_0 + Z_1$	+192	+230	+252	+258	+252	+234	+207	+174	+184	+95	+52	+4	2.258	4.054	-1.796	1 158	2 070	Below floor.
6-7	Z_1	-715	-650	-585	-520	-455	-390	-325	-260	-195	-130	-65	0						
	$Z = Z_0 + Z_1$	+126	+170	+198	+210	+210	+198	+177	+160	+116	+83	+46	+4	1.928	2.725	-0.797	894	1 264	
8-9	Z_1	-804	-730	-657	-584	-511	-438	-365	-292	-219	-146	-73	0						
	$Z = Z_0 + Z_1$	+37	+90	+126	+146	+154	+150	+137	+118	+92	+67	+38	+4	1.824	2.621	-0.797	725	1 042	
10-11	Z_1	-900	-819	-737	-655	-573	-491	-409	-327	-246	-164	-82	0						
	$Z = Z_0 + Z_1$	-59	+1	+46	+75	+92	+97	+93	+83	+65	+49	+29	+4	1.818	2.615	-0.797	663	954	
12-13	Z_1	-1 033	-939	-845	-751	-657	-563	-470	-376	-282	-188	-94	0						
	$Z = Z_0 + Z_1$	-192	-119	-62	-21	+8	+25	+32	+34	+29	+25	+17	+4	2.155	2.952	0.797	687	940	
14-15	Z_1	-1 239	-1 126	-1 014	-901	-788	-676	-563	-450	-338	-225	-113	0						
	$Z = Z_0 + Z_1$	-398	-306	-231	-171	-123	-88	-61	-40	-27	-12	-2	+4	3.341	4.141	-0.797	855	1 069	
16-17	Z_1	-1 645	-1 496	-1 346	-1 197	-1 047	-897	-748	-598	-449	-299	-150	0						
	$Z = Z_0 + Z_1$	-804	-676	-563	-467	-382	-309	-246	-188	-138	-86	-39	+4	6.152	6.949	-0.797	1 082	1 222	
18-19	Z_1	-3 086	-2 805	-2 525	-2 244	-1 964	-1 683	-1 403	-1 122	-842	-561	-281	0						
	$Z = Z_0 + Z_1$	-2 245	-1 985	-1 742	-1 514	-1 310	-1 095	-901	-712	-531	-348	-170	+4	16.880	17.127	-0.797	1 285	1 348	
20-21	Z_1	+5 721	+5 201	+4 681	+4 101	+3 603	+3 121	+2 600	+2 080	+1 560	+1 040	+520	0						
	$Z = Z_0 + Z_1$	+6 562	+6 021	+5 464	+4 891	+4 328	+3 709	+3 102	+2 490	+1 871	+1 258	+631	+4	45.410	46.216	-0.797	1 482	1 408	
22-23	Z_1	179	-163	147.0	130	114	98.4	82	65.0	49.0	33	16.0	0						
	Z_2	136.4	106.7	81.5	61.3	45.4	33.2	23.6	16.1	11.1	5.8	1.4	+1.2						
	$Z = Z_0 + Z_1 + Z_2$	+526	+550	+554	+539	+506	+457	+396	+320	+251	+174	+94	+5	4.196	6.033	1.837	879	612	

Stress (22-23) $= -0.2858\,(\eta_H - 0.17094\, m + 2.405\, \eta_z)$.
Stress (22-23)' $= -0.2858\,(\eta_H - 0.01605\, m + 2.405\, \eta_z) = -0.2858\,(Z_0 + Z_1 + Z_2)$.

TABLE 17.—DETERMINATION OF DEAD-LOAD STRESSES FOR CASE II.

Panel Length, $\lambda = 42.5$ ft. All Loads and Stresses Given in Units of 1 000 Lb.

PANEL POINTS.	(1) 0-1	(2) 2-3	(3) 4-5	(4) 6-7	(5) 8-9	(6) 10-11	(7) 12-13	(8) 14-15	(9) 16-17	(10) 18-19	(11) 20-21	(12) 22-23
Loads at top chord	+ 188	+ 187	+ 174	+ 157	+ 160	+ 155	+ 135	+ 140	+ 140	+ 140	+ 137	+ 130
Loads at floor height	+ 308	+ 261	+ 234									
Loads at bottom chord	+ 324	− 143.23	− 196.23	− 42.23	− 92.23	− 97.23	− 143.23	− 164.23	− 165.23	− 175.23	− 182.23	− 170.23
Total	+ 820	+ 304.77	+ 211.77	+ 114.77	+ 67.77	+ 57.77	− 8.23	− 24.23	− 25.23	− 35.23	− 45.23	− 40.23
Shear	+ 578.47	+ 273.70	+ 61.93	− 52.84	− 120.61	− 178.38	− 170.15	− 145.92	− 120.69	− 85.46	− 40.23	
Moment = M	0	+ 578	+ 852	+ 914	+ 861	+ 741	+ 562	+ 392	+ 246	+ 126	+ 40	0
TOP CHORD.	1-3	3-5	5-7	7-9	9-11	11-13	13-15	15-17	17-19	19-21	21-23	23-23A
Stress, Case II, $-\frac{M\lambda}{r}$	− 228	− 408	− 523	− 558	− 529	− 441	− 336	− 228	− 124	− 41	0	0
BOTTOM CHORD.	0-2	2-4	4-6	6-8	8-10	10-12	12-14	14-16	16-18	18-20	20-22	22-22A
Stress, Case II, $\frac{M\lambda}{r}$	0	+ 276	+ 477	+ 578	+ 587	+ 549	+ 452	+ 341	+ 229	+ 125	+ 41	0
$\frac{L}{s_b} = \frac{\text{Bottom chord stress}}{\text{Length of member, in feet}}$	0	+ 5.12	+ 9.17	+ 11.41	+ 12.10	+ 11.69	+ 9.89	+ 7.65	+ 5.25	+ 2.90	+ 0.97	0
$\frac{L'}{s_{b'}} - \frac{L}{s_b}$	+ 5.12	+ 4.05	+ 2.24	+ 0.69	− 0.41	− 1.80	− 2.24	− 2.40	− 2.35	− 1.93	− 0.97	0
DIAGONAL.	1-2	3-4	5-6	7-8	9-10	11-12	13-14	15-16	17-18	19-20	21-22	23-22
Stress $= s_d \left[\frac{L'}{s_{b'}} - \frac{L}{s_b}\right]$	+ 572	+ 365	+ 170	+ 47	− 26	− 109	− 131	− 137	− 132	− 109	− 55	0
VERTICAL.	0-1	2-3	4-5	6-7	8-9	10-11	12-13	14-15	16-17	18-19	20-21	22-23
$-s_v \left[\frac{L'}{s_{b'}} - \frac{L}{s_b}\right]$	Reaction for Case II = 1 400	− 716.80	− 457.37	− 208.17	− 55.20	+ 29.18	+ 114.17	+ 127.28	+ 123.05	+ 110.10	+ 84.04	+ 40.09
$C\frac{L'}{s_{b'}}$		+ 17.07	+ 30.56	+ 38.02	+ 40.29	+ 38.96	+ 32.96	+ 25.50	+ 17.50	+ 9.67	+ 3.23	0
Lower panel point concentration	− 324	− 143.23	− 196.23	− 42.23	− 92.23	− 97.23	− 143.23	− 164.23	− 165.23	− 175.23	− 182.23	− 170.23
Stress in vertical below floor	− 1076	− 843	− 623	− 212	− 107	− 29	+ 4	− 12	− 25	− 55	− 95	− 130
Stress in vertical above floor	− 768	− 582	− 389									



Length of $AB = s_b$

Length of $BC = s_{b'}$

Length of $DB = s_d$

Length of $AD = s_v$

Stress in $AB = L$

Stress in $BC = L'$

Ordinate $C = 3\frac{1}{4}$ ft.

To simplify the calculation, these ordinates were assumed as multiplied by the value, $\frac{r}{y}$; in other words, the ordinates, $A_1\ A_2$ and $B_1\ B_2$, were made equal to $\frac{x}{y}$ and $\frac{x'}{y}$, respectively (Fig. 55). The intermediate ordinates, Z_1, of the lines, $A_1\ C_2$ and $C_2\ B_1$, were then obtained by simple proportion from the ordinates, $A_1\ A_2$ and $B_1\ B_2$. (Tables 13, 14, 15, and 7.) These ordinates, Z_1, were then added algebraically to the influence ordinates, Z_0, for the horizontal reaction (polygon, $A_1\ C_1\ B_1$), in order to get the influence ordinates, Z, for the two-hinged arch condition.

To obtain any stress, the sum of the influence ordinates, Z, finally had to be multiplied by the coefficient, $\frac{y}{r}$.

3.—Live-Load Stresses.

The assumed live load is 6 000 lb. per lin. ft. of track, or 12 000 lb. per lin. ft. of truss, which gives a full panel load per truss of

$$12\,000 \times 42.5 \text{ ft.} = 510\,000 \text{ lb.}$$

To determine the maximum live-load stress in any member, the influence ordinates of the same sign were added (Columns 31 and 32, Tables 13, 14, 15, and 16), proper corrections being made for partial panel loads at the end vertical and at panel points adjacent to the zero points of the influence line.

The sum of the influence ordinates was then multiplied by the coefficient, $\frac{y}{r}$, multiplied by the panel load, 510 000 lb. (Columns 34 and 35, Tables 13, 14, 15, and 16).

4.—Dead-Load Stresses.

The arch was erected so as to act as three-hinged for all the dead load except the concrete and timber flooring, ballast, and tracks, which were placed after the trusses had been converted into two-hinged arches.

For the three-hinged arch, the stresses were determined as follows:

First, the horizontal reaction was determined for the actual panel concentrations. Then, a uniform load was determined which would cause the same horizontal reaction. For this uniform load (16 000 lb. per lin. ft., or 680 000 lb. per panel per truss), the stresses were determined in the bottom chord members (Case I). As the bottom chord panel points are on a parabola, no other members are stressed for this case, and the stress in any bottom chord is equal to the horizontal reaction multiplied by the ratio between the length of the member and the panel length.

TABLE 18—DEAD-

All Stresses Given in Units of 1 000 lb.

BOTTOM CHORD.	0–2	2–4	4–6	6–8
Case I	— 11 457	— 11 023	— 10 616	— 10 239
Case II	0	+ 276	+ 477	+ 573
Total case, I + II	— 11 457	— 10 747	— 10 139	— 9 666
Case III	— 5 008	— 4 712	— 4 406	— 4 101
Total case, I + II + III	16 465	— 15 459	— 14 545	— 13 767
TOP CHORD.	1–3	3–5	5–7	7–9
Case I	0	0	0	0
Case II	— 223	— 408	— 523	— 558
Total case, I + II	— 223	— 408	— 523	— 558
Case III	— 84	— 199	— 340	— 488
Total case, I + II + III	— 307	— 607	— 863	— 1 046
DIAGONALS.	1–2	3–4	5–6	7–8
Case I	0	0	0	0
Case II	+ 572	+ 365	+ 170	+ 47
Total case, I + II	+ 572	+ 365	+ 170	+ 47
Case III	+ 216	+ 230	+ 224	+ 215
Total case, I + II + III	+ 788	+ 595	+ 394	+ 262
VERTICALS.	0–1 Below floor.	2–3 Below floor	4–5 Below floor	6–7
Case I	0	0	0	0
Case II	— 1 076	— 843	— 623	— 212
Total case, I + II	— 1 076	— 843	— 623	— 212
Case III	— 374	— 561	— 570	— 230
Total case, I + II + III	— 1 450	— 1 403	— 1 193	— 442
VERTICALS.	Above floor	Above floor	Above floor	
Case II	— 768	— 582	— 389	....
Total case, I + II	— 768	— 582	— 389	....
Case III	— 218	— 244	— 253	....
Total case, I + II + III	— 986	— 826	— 642	....

Horizontal Reaction for Case I = 8 672 900 lb. Bottom Chord Stress for Case I = Horizontal

LOAD STRESSES.

8–10	10–12	12–14	14–16	16–18	18–20	20–22	22–22′
— 9 896 + 587	— 9 585 + 549	— 9 315 + 452	— 9 089 + 341	— 8 910 + 229	— 8 779 + 125	— 8 700 + 41	— 8 673 0
— 9 309	— 9 036	— 8 863	— 8 748	— 8 681	— 8 654	— 8 659	— 8 673
— 3 812	— 3 532	— 3 259	— 3 008	— 2 785	— 2 604	— 2 481	— 2 305
— 13 121	— 12 568	— 12 122	— 11 756	— 11 466	— 11 258	— 11 140	10 978
9–11	**11–13**	**13–15**	**15–17**	**17–19**	**19–21**	**21–23**	**23–23′**
0 — 529	0 — 441	0 — 336	0 — 228	0 — 124	0 — 41	0 0	0 0
— 529	— 441	— 336	— 228	— 124	— 41	0	0
— 636	— 793	— 951	— 1 103	— 1 234	— 1 332	— 1 384	— 1 287
— 1 165	— 1 234	— 1 287	— 1 331	— 1 358	— 1 373	— 1 384	— 1 287
9–10	**11–12**	**13–14**	**15–16**	**17–18**	**19–20**	**21–22**	**23–22′**
0 — 26	0 — 109	0 — 131	0 — 137	0 — 132	0 — 109	0 — 55	0 0
— 26	— 109	— 131	— 137	— 132	— 109	— 55	0
+ 222	+ 227	+ 225	+ 213	+ 185	+ 140	+ 77	— 138
+ 196	+ 118	+ 94	+ 76	+ 53	+ 31	+ 22	— 138
8–9	**10–11**	**12–13**	**14–15**	**16–17**	**18–19**	**20–21**	**22–23**
0 — 107	0 — 29	0 + 4	0 — 12	0 — 25	0 — 55	0 — 95	0 — 130
— 107	— 29	+ 4	— 12	— 25	— 55	— 95	— 130
— 197	— 181	— 158	— 127	— 87	— 39	+ 15	+ 166
— 304	— 210	— 154	— 139	— 112	— 94	— 80	+ 36
....	....	....	....	....	....	....	....
....	....	....	....	....	....	....	....
....	....	....	....	....	....	....	....
....	....	....	....	....	....	....	....

Reaction $\times \dfrac{\text{Length of Member}}{\text{Panel Length}}$. + Denotes Tension. – Denotes Compression.

Then the stresses were determined for the difference between the actual and the uniform load (Case II). As, for both the actual and the uniform loads, the horizontal reactions are the same, that is, their difference is zero, it was possible to determine the stresses for Case II as for a simple span. The stresses in the chord members were determined from the bending moments. The stresses in the web members were then obtained by resolving stresses at the lower panel points. With the notations given in the diagram on Table 17 (Plate XXXV) we have, stress in diagonal $= s_d \left(\frac{L'}{s_b'} - \frac{L}{s_b}\right)$, stress in vertical below floor $= - s_v \left(\frac{L'}{s_b'} - \frac{L}{s_b}\right) + C \frac{L'}{s_b'} +$ panel load at bottom chord.

The stress in the vertical above the floor is found by deducting the panel load at the floor height from the stress below the floor.

For the two-hinged arch (Case III), the stresses were determined from the influence ordinates in a manner similar to that described for live loads. For panel concentrations, see Plate XX.

Finally, the stresses for Cases I, II, and III were combined to obtain the total dead-load stresses (Table 18).

5.—Temperature Stresses.

Assuming the arch bearings as free to move longitudinally, a horizontal force of unity applied at each hinge causes a change in the span length equal to $\Sigma \Delta l$. A change in temperature of $t = 72°$ Fahr. causes a change in the span length equal to $\varepsilon\, t\, l = \frac{1}{150\ 000} \times 72 \times 977.5 \times 12 = 5.63$ in.

The horizontal reaction due to a change of temperature of 72° Fahr., therefore, is

$$H_t = \frac{\varepsilon\, t\, l}{\Sigma \Delta l} = 215\ 460 \text{ lb.}$$

The temperature stress in any member is then found as the product of H_t with the corresponding value, $S_1 = \frac{y}{r}$ (Table 19).

TABLE 19.—TEMPERATURE STRESSES FOR A VARIATION OF $\pm$ 72° FAHR.

Stresses Given in Units of 1 000 lb.

TOP CHORD.	1-3	3-5	5-7	7-9	9-11	11-13	13-15	15-17	17-19	19-21	21-23	23-23′
Temperature stress.......	$\pm$ 72	$\pm$ 170	$\pm$ 291	$\pm$ 416	$\pm$ 543	$\pm$ 677	$\pm$ 811	$\pm$ 940	$\pm$ 1 052	$\pm$ 1 136	$\pm$ 1 180	$\pm$ 1 157
BOTTOM CHORD.	0-2	2-4	4-6	6-8	8-10	10-12	12-14	14-16	16-18	18-20	20-22	22-22′
Temperature stress......	$\mp$ 285	$\mp$ 363	$\mp$ 463	$\mp$ 573	$\mp$ 683	$\mp$ 802	$\mp$ 925	$\mp$ 1 049	$\mp$ 1 166	$\mp$ 1 269	$\mp$ 1 349	$\mp$ 1 416
DIAGONAL.	1-2	3-4	5-6	7-8	9-10	11-12	13-14	15-16	17-18	19-20	21-22	23-22
Temperature stress......	$\mp$ 184	$\mp$ 196	$\mp$ 192	$\mp$ 183	$\mp$ 190	$\mp$ 194	$\mp$ 192	$\mp$ 181	$\mp$ 158	$\mp$ 119	$\mp$ 66	$\pm$ 29
VERTICAL.	0-1	2-3	4-5	6-7	8-9	10-11	12-13	14-15	16-17	18-19	20-21	22-23
Temperature stress......	$\pm$ 186	$\pm$ 208	$\pm$ 216	$\pm$ 196	$\pm$ 168	$\pm$ 154	$\pm$ 134	$\pm$ 108	$\pm$ 74	$\pm$ 33	$\mp$ 13	$\mp$ 81

Horizontal reaction, H_t (for $\pm$ 72° Fahr.) = 215.460.
Stress in member = horizontal reaction $\times$ S_1.
For values of S_1, see Tables 7 and 8.

+ Denotes tension.
— Denotes compression.

APPENDIX B

Financing, and Franchise of the New York Connecting Railroad Company.

A General Statement, Furnished by the Railroad Company.

The scope and cost of this extraordinary work and the great possibilities for transportation service which it opens up, justify a short record for public information as to the principal facts concerning the company under whose powers it was built, its incorporation, and its financing.

The railroad was originally conceived in 1892 by Oliver Barnes and Gustav Lindenthal, Members, Am. Soc. C. E., on practically the same location as that on which it was built.

The Certificate of Incorporation of The New York Connecting Railroad Company was filed and recorded in the office of the Secretary of State of the State of New York on April 21st, 1892, and provided for a steam railroad of about 10 miles in length, the termini of which were to be "in Westchester County, east of the Bronx River, and in the City of Brooklyn."

Among the incorporators were: Oliver W. Barnes, Frank M. Clute, Alfred P. Boller, Charles Macdonald, and Thomas S. King; the capital stock was placed at one thousand shares, par $100 000, all preferred stock.

In April, 1902, The Pennsylvania Railroad Company completed the purchase of the entire outstanding stock of the Connecting Company, and, in accordance with a prior understanding with The New York, New Haven and Hartford Railroad Company, a short time thereafter, sold one-half of this stock to that Company.

On June 11th, 1903, application for the franchise required from the City of New York was made by the Company to the Board of Rapid Transit Railroad Commissioners for that City, predecessor of the present Public Service Commission for the First District of the State of New York, and a tentative franchise was granted by that Board on June 23d, 1904, subject to the approval of the Board of Aldermen and the Mayor. After consideration by the Board of Aldermen for nearly a year, however, the proposed franchise was, on April 18th, 1905, returned to the Board of Rapid Transit Railroad Commissioners "disapproved."

On November 17th, 1905, the application to the Board of Rapid Transit Railroad Commissioners for a franchise was renewed, and, as a result of negotiations extending over a year, the franchise now held

by the Company, dated February 14th, 1907, was granted. This Certificate was accepted by the Company under date of February 28th of that year; was approved by the Board of Estimate and Apportionment of the City on March 8th (by amendments to the City Charter and Rapid Transit Act of the State, the duty and power to confirm franchises of this character had been transferred since the previous application from the Board of Aldermen to the Board of Estimate and Apportionment); was approved by the Mayor on March 14th, and, later, the Certificate was filed in the office of the Secretary of State of New York and in the offices of the Clerks of the Counties of New York, Queens, and Kings.

In addition to fixing the center line of the proposed railroad, the franchise, among other requirements, prescribed that the consent of the owners of one-half in value of the property bounded on the portions of streets crossed by the line, to the construction and operation thereof, should be obtained within one year from the acceptance of the franchise by the Company; that construction was to commence within 3 months after filing said consents with the Rapid Transit Board; and that construction was to be completed and the railroad in operation within 5 years after the commencement of construction. Provision was made in each case for extensions of time under certain conditions. The motive power prescribed to be used was steam, with the right to the Railroad Company to substitute electricity therefor, and, in addition, the Rapid Transit Board reserved the right, in the event of the use of steam constituting a nuisance or becoming dangerous to residents along the route, to require a change to electricity, or other motive power not less convenient to the public, within a period not to be less than 3 years after notice by the Board. The franchise fixes the rentals to be paid by the Company for the first period of 25 years from 2 years after obtaining the required consent of property owners (except for the use of Wards and Randalls Islands, in which case the rental was payable from the date of first occupation) and provides for a readjustment thereof at the end of said period and every 25 years thereafter. Said payments (as amended by order of the Public Service Commission in connection with the shortening of the franchise route of the Company from Knickerbocker Avenue to Fresh Pond Junction, hereinafter dealt with) are as follows: (*a*) For the right to construct and operate across streets and other public property other than Wards and Randalls Islands, \$23 925 per annum for the first 10 years of said 25-year period, and \$47 850 for the next 15 years; (*b*) \$100 per annum for the right to cross the East River between bulkhead lines; and (*c*) for the use of ground on Wards and Randalls Islands occupied, permanently and temporarily, during the

period of construction, by the abutments, piers, and other supports of the bridges and elevated structures, and for the use of portions of overhead space above the islands occupied by such bridges or elevated structures or for any other purpose, such annual payments as agreed upon by the Company and the Board of Commissioners of the Sinking Fund of the City, or other authorities in control of the islands. In addition to said rentals, the Company was compelled to file with the Comptroller of the City, within 60 days of the approval of the franchise by the Mayor, a bond in the sum of \$50 000, executed by it and by The Pennsylvania Railroad Company and The New York, New Haven and Hartford Railroad Company, as security for the performance by the Railroad Company of the terms and conditions of the franchise, especially with respect to the annual payments to the City, and had to pay a bonus of \$110 000 to the City within 60 days after the necessary consents of property owners had been obtained and before construction work was commenced.

On April 16th, 1907, the stockholders approved of an increase in the capital stock from \$100 000, Preferred, to \$3 000 000, Common, the holders of the Preferred stock having agreed to exchange their stock, par for par, for Common stock. The increase was approved by the Board of Railroad Commissioners on May 31st, and, as of June 1st, the exchange of Preferred stock was made and \$2 816 400 of the Common stock was issued, half and half, to the Pennsylvania Railroad Company and The New York, New Haven and Hartford Railroad Company, to take up a like amount of advances to the Connecting Company. Subsequently, the remainder of the authorized stock was issued, the entire \$3 000 000 thereof being held by The Pennsylvania Railroad Company and The New York, New Haven and Hartford Railroad Company in one-half proportions, all of the stock having been fully paid for at par.

The necessary consents of property holders to the construction of the line were filed with the Public Service Commission for the First District of the State of New York on July 20th, 1910, the said Commission having succeeded to the rights and powers of the Board of Rapid Transit Commissioners under the law. The original period fixed by the franchise in which to obtain said consents expired on February 28th, 1908, but, by certificate of the Commission dated February 3d, 1908, and March 5th, 1910, the time was extended to August 31st, 1910.

The period prescribed in the franchise for the completion of the railroad, including the Hell Gate Bridge, expired on October 20th, 1915, but the time was extended by the Commission to December 31st, 1917, and the same date was fixed by General and Special Laws of the State of New York.

The railroad as constructed conforms to the route prescribed in the franchise, except for a slight change of alignment between the north side of Wards Island and 132d Street, in the Borough of The Bronx, a change in grade between Broadway and Calamus Avenue, in the Borough of Queens, and a change in the southern terminus of the line from the Brooklyn Borough line to Fresh Pond Junction. The two changes first mentioned were approved by the Public Service Commission on July 19th, 1912, and maps covering the revisions were filed in New York and Kings Counties on July 23d, 1912, and in Queens County on July 24th, 1912; and the shortening of the franchise route to Fresh Pond Junction was approved by said Commission on June 7th, 1915; approved by the Board of Estimate and Apportionment on July 29th; by the Mayor on July 30th; and a map showing the modification was filed in the office of the Secretary of State of New York on August 13th, 1915, and also in the offices of the Clerks of the Counties of New York, Queens, Kings, and The Bronx. The approval of the War Department to the plans for the Hell Gate, Little Hell Gate, and Bronx Kill Bridges was granted by Certificate dated June 22d, 1906, and the changes in the plans for the two last mentioned bridges necessitated by the change in the route of the road across Randalls Island, hereinbefore mentioned, were approved by Certificate of April 4th, 1912.

In accordance with the requirements of the Franchise, the Company, on May 11th, 1907, submitted the plans for the Hell Gate Bridge to the Municipal Art Commission of the City of New York for its approval. By certificate of June 27th of that year, such approval was deferred, the Commission objecting to the proposed decorative treatment of the towers of the bridge. On May 29th, 1911, the plans were re-submitted to the Commission, the architectural features of the towers having been altered to meet the views of that body, and, by resolution of June 13th, 1911, the necessary approval was granted, a certified copy of which was, on August 7th, 1911, forwarded to the Public Service Commission for file, and receipt was acknowledged by it under date of August 8th.

The cost of constructing The New York Connecting Railroad, over and above the \$3 000 000 received from the sale of capital stock, was financed through the issue and sale of bonds covered by a first mortgage, dated May 31st, 1913, executed to the Guaranty Trust Company of New York, Trustee, for \$30 000 000, which was approved by the Public Service Commission by Order dated November 14th, 1913. Of these bonds, \$24 000 000 have been issued to this date and about \$1 500 000 additional will be issued; all are guaranteed, principal and interest, by The Pennsylvania Railroad Company and The New York, New Haven and Hartford Railroad Company.

The present Directors of the Company are:

Samuel Rea, W. W. Atterbury, George D. Dixon, W. H. Myers, and A. J. County.	Representing The Pennsylvania Railroad Company.
Howard Elliott, B. Campbell, E. G. Buckland, J. M. Tomlinson, and A. R. Whaley.	Representing The New York, New Haven and Hartford Railroad Company.

Mr. Samuel Rea (President of The Pennsylvania Railroad Company) is President of The New York Connecting Railroad Company; Mr. Howard Elliott (President of The New York, New Haven and Hartford Railroad Company) is Vice-President; and Mr. A. J. County (Vice-President of The Pennsylvania Railroad Company) is Assistant to the President of The New York Connecting Railroad.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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PAPERS AND DISCUSSIONS

STRESS MEASUREMENTS ON THE HELL GATE ARCH BRIDGE

By D. B. Steinman, Assoc. M. Am. Soc. C. E.

To be Presented November 21st, 1917.

Synopsis.

The magnitude of the Hell Gate Arch Bridge and its interesting construction features suggested the desirability of utilizing the bridge as an instrument for scientific research by conducting a series of stress measurements extending through all the different stages of erection until the structure was completed. As a result of these observations, the Hell Gate Arch is probably the only structure ever built in which the true stress conditions are known from experimental determination.

In order to give this investigation its proper setting and perspective, there is presented, as an introduction, a résumé of previously published measurements of stresses in bridges, with a brief statement of the most significant results obtained. From these findings, particularly with reference to secondary stresses, conclusions are drawn by which to improve the design of railway viaducts and bridges. The record, on the whole, indicates the scarcity of such observations in the past, and serves to emphasize the value and importance of undertaking more investigations of this character in order to confirm or correct the results of theoretical analysis. This experimental verification of

Note.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and, when finally closed, the papers, with discussion in full, will be published in *Transactions*.

stress conditions is particularly desirable in fields previously unexplored, or wherever there are special features which may produce uncertain variations from calculated conditions.

The original objects in view in undertaking these stress measurements were: to follow up the stresses in arch and back-stays as critical erection stages were approached; to check and control certain critical operations in the erection; to check the stresses in the completed indeterminate structure, in order to detect any variation from assumed conditions; and to determine the true secondary stresses for comparison with the calculated values.

The paper includes a brief description of the instruments, the method of operation, and the precautions required for accurate work. This is followed by a full presentation of the results, including a tabulated comparison of the calculated and measured primary and secondary stresses.

From the results of the measurements and comparisons, conclusions are deduced on the following subjects:

1.—The precision attainable with the instrument, and the reliability of this method of measuring stresses;

2.—The probable error of calculated stresses, indicating the futility of excessive precision in their computation;

3.—The extreme variations of fiber stress in a member, representing the combined effect of all known and unknown secondary strains;

4.—Safe working values for the design of bridge members, leaving a margin below the elastic limit for extreme variations from calculated stresses;

5.—Relative values of calculated and measured secondary stresses (it is shown that the latter are generally lower);

6.—The effect of the erection operations on the secondary stresses;

7.—The efficacy of the three-face joints in the lower chord (Fig. 6), which were provided in order to produce a hinge action at the panel points during erection and to concentrate a larger part of the direct stress in the middle third of the cross-section, it having been demonstrated that this novel splicing feature has accomplished the desired objects, thereby reducing the secondary and direct stresses in the outer fibers;

8.—The re-distribution of stresses and the release of secondary strains at a splice during the replacing of drift-pins by rivets;

9.—The comparative freedom of the arch truss from secondary stresses; and

10.—The final extreme fiber stresses as compared with the calculated values.

Special acknowledgment is due to Gustav Lindenthal, M. Am. Soc. C. E., the Chief Engineer of the bridge, who undertook to make these measurements in furtherance of engineering science.

Résumé of Previously Published Stress Measurements.

Since the early days of bridge building, when empirical methods of design prevailed, theoretical analysis has made rapid advances, and has now far outstripped experimental verification. There is a growing movement, however, as shown by the modern testing of large compression members, and in load tests and strain measurements on bridges, to supplement the results of analysis with experimental observation in order to confirm or correct the former, or to reveal unsuspected conditions.

The first measurements of the actual stresses in trusses were made by Professor Fränkel* in 1883. He applied his extensometer to the experimental determination of secondary stresses, and his results produced such uneasiness among German engineers that some of them abandoned the riveted truss in favor of pin connections.

In 1899 Mesnager published an account of the measurement of stresses in a bridge of 180 ft. span, on the Orléans Railway in France, with a discussion† of the results. It was a ten-panel, Pratt truss bridge carrying a single track. The stresses were measured only in the web members, and for a train load of two locomotives and four cars, covering the entire span. The extensometers, devised by M. Rabut, were applied at four points of the cross-section near each end of a member. The results showed the average stresses in the members to be in fair agreement with the calculated primary stresses. There were considerable differences, however, between the stresses on opposite sides of a member. In the posts, secondary stresses were found amounting to 200% of the direct stress, involving a complete reversal of stress on one side of the member. In the diagonals the greatest

* Versuche mit dem Dehnungszeichner", *Der Civilingenieur*, 1883.

† "Les Fatigues Réelles et Fatigues Calculées dans un Pont à Grandes Mailles", *Annales des Ponts et Chaussées*, 1899, II.

secondary stresses were 45% of the direct stress. The foregoing large secondary stresses do not form the basis for any general conclusions, as they were mainly due to peculiarities in the design of the structure.

Mesnager did not undertake a computation of the secondary stresses in the test bridge, so that no comparison of actual with calculated values is afforded.

In 1901 M. Rabut described* a series of stress measurements which had been made on the bridges of the Orléans Railway. The experiments covered small plate-girder and pony-truss spans. In some plate girders, in reference to which apprehension had been aroused by excessive calculated stresses, the measured stresses were found to be considerably lower. Plate girders supporting longitudinal ties and rails were found to act as beams with fixed ends, with a large reduction of stress at the center, and reversed stresses at the ends. In pony trusses, secondary stresses as high as 30% appeared on the outside of the top chords, caused by the trusses bending inward near the center of the bridge as a result of the floor-beam deflections. The bottom chords of through bridges showed smaller stresses than the top chords, on account of the former being partly relieved by the stringers and the bottom laterals taking part in the elongation. All floor-beams were found to act nearly as simple beams, the end constraint being negligible. Stringers acted as simple beams for a load on one side of the floor-beam, and as beams with fixed ends for symmetrical loading.

The results of these experiments demonstrated the necessity of detailing stringers for full negative bending moments where they frame into floor-beams; the desirability of using deep stringers to avoid high torsional stresses in the floor-beams from one-sided loading; and the importance of using deep floor-beams and verticals comparatively slender in the transverse direction in order to minimize the bending stresses arising from the floor-beam deflections.

In 1905 and 1906, W. Gehler conducted a series of tests and measurements† on a railway bridge of 128 ft. span at Elsterwerda, Saxony. It was a ten-panel, Pratt truss, skew-bridge; and the loading

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* "Conference sur l'Experimentation des Ponts", *Annales des Ponts et Chaussées*. 1901, III.

† "Nebenspannungen eiserner Fachwerkbrücken", Berlin, 1910.

consisted of two tank cars placed near the middle of the span. The vertical deflections at the lower panel points were measured with Fränkel-Leuner deflectometers, and the rotation angles of the same points were observed with extreme care by using highly sensitive spirit levels. The measurements agreed closely with the values previously calculated, except for some small variations which were traced to the stiffness of the track and the effect of the skew arrangement of the trusses. The results, on the whole, afforded a valuable proof of the remarkably close agreement attainable between theoretical computations and actual conditions in a bridge structure.

Gehler had figured the secondary stresses in the trusses from the theoretical vertical and angular deflections of the lower panel points; and he regarded this check on these deflections as a verification of the calculated secondary stresses. He recommends this procedure as an experimental method for determining the actual secondary stresses in a truss.

To make the foregoing investigation complete, some direct stress measurements were made with Fränkel-Leuner extensometers,* which give automatic graphic records of the varying stresses. On account of the unsatisfactory precision of these instruments, the results were so erratic that Gehler concluded that extensometers are not suitable for the measurement of secondary stresses.

In 1907-09 a sub-committee of the American Railway Engineering Association, consisting of F. E. Turneaure, M. Am. Soc. C. E., C. L. Crandall, M. Am. Soc. C. E., the late C. H. Cartlidge, M. Am. Soc. C. E., and the late C. C. Schneider, Past-President, Am. Soc. C. E., conducted a series of tests on a large number of plate-girder and truss bridges, ranging in span from 30 to 440 ft. They used special test trains, and measured the deflection at the center of the span and the strains in the various kinds of members. The object of the tests was, not to determine secondary stresses, nor to compare calculated and measured primary stresses, but to ascertain the relative amounts of the resulting deflections and strains for various speeds of the moving loads, in order to establish the proper provision to be made for impact stresses. The results and conclusions are contained in the report of the Committee.†

* *Der Civilingenieur*, 1882, p. 200.

† *Bulletin No. 125*, Am. Ry. Eng. Assoc., July, 1910.

In 1911 the Sub-Committee of the American Railway Engineering Association made a theoretical and experimental study of secondary stresses in truss bridges. In several bridges, of both riveted and pin-connected types, the secondary stresses at selected points were measured with extensometers. In the case of a 105-ft. pony Warren truss, a comparison was made of the calculated and measured secondary stresses in the members. The results are published in the report* of the Committee.

With the exception of the 105-ft. span last mentioned, the writer knows of no published comparison between measured and calculated secondary stresses.

Gehler's work was only a partial check on the calculations of secondary stresses, as he merely measured functions, namely, panel point deflections and rotations, from which the actual secondary stresses had to be computed.

All the published stress measurements were made for loads on existing structures. They afford no information as to the strains produced by dead load or the effect of the erection on the secondary stresses.

None of the published measurements was made on indeterminate structures or on any structure exceeding 440 ft. in span.

Reasons for Taking the Measurements.

The magnitude of the Hell Gate Arch Bridge and its unusual structural features suggested to Mr. Lindenthal the desirability of utilizing the structure for scientific research. By preparing the members for extensometer measurements and taking initial readings before erection, the huge arch was converted into an instrument for the experimental study of the true stress conditions in a structure.

The trusses of the bridge are doubly indeterminate: they are two-hinged arches, and they have redundant diagonals in the center panel. It has often been advanced, as an objection to the use of indeterminate bridge types, that the computations are highly theoretical and the actual stress conditions are uncertain. It is hardly necessary to remark that this argument should have little weight under modern methods of design and construction. As an answer to such objections in the future, it appeared to be of interest to demonstrate, by measure-

* *Bulletin No. 163*, January, 1914.

ment, the identity of calculated and actual stresses in the finished structure.

Another possible purpose of extensometer measurements is to supply an added safeguard for a structure during erection by measuring the stresses at critical points. The stresses in the various members of the arch and in the back-stays were measured in the successive stages as the trusses were built out from shore, panel by panel, to their junction at mid-span. When any of the stresses were nearing their maximum values, they were closely watched until the critical stage was passed.

In one instance, one of the eye-bars in the forward stay was found to have a stress 55% higher than the average stress in the group of eye-bars. This appeared in an early stage of the erection, when the stresses were low. As the erection progressed, however, and as was to be expected, the difference in stress gradually diminished until, in the last stage, all the eye-bars were found to be uniformly strained to nearly 20 000 lb. per sq. in.

Another application of the extensometer was to provide a check in some of the critical operations in the erection of the structure. After seven panels had been erected, the hydraulic jacks at the tops of the erection towers were operated to put the forward stay in tension and release the lower stay. The completion of this operation was checked by measurements of the stresses in both stays. When this jacking was performed on the Wards Island side, a small compression was found in the lower stay, indicating that the jacking had gone too far; the jacks were then lowered slightly until the stress was reduced to zero.

After the trusses met at mid-span, the lowering of the jacks to close the arch was closely followed by measurement of the diminishing stress in the forward stay. Measurements were also made in the central chord member, in order to make sure that there was no eccentricity of bearing at the junction of the two half-arches.

The final operation consisted in closing the top chord at the crown in order to convert the structure into a two-hinged arch. A large bolt, connecting across the joint, was provided in order to hold the members together until the drilling of holes and riveting could be completed. This bolt had to be adjusted, by a set of nuts, to an initial condition corresponding to zero stress at 60° Fahr. The exten-

someter was used to control this operation and to check the subsequent stresses in the top-chord member.

The final object of the extensometer measurements was to provide a comparison between the calculated and the actual secondary stresses in the structure. In addition to the general scientific value of such a comparison, there were special conditions which called for a determination of the true stresses. These conditions were the cantilever method of erection, the use of drift-pins for temporary connection and their subsequent replacement by rivets, the three-faced butt-joints in the lower chord, and the unprecedented dimensions and form of cross-section. These features, separately and combined, modify the secondary stresses materially and render it extremely difficult, if not impossible, to arrive at the true secondary stresses by calculation.

The Instrument and the Method of its Operation.

The instrument used for the stress measurements on the Hell Gate Arch Bridge (Fig. 1) was a 20-in. strain gauge, designed by Mr. James E. Howard. This instrument is essentially a micrometer caliper. The measuring points, made of hard steel and conical in form, are attached to the barrel and the rod, respectively; and the distance between the two points is measured by a micrometer contact screw at one end of the barrel. This screw reads, by a circumferential vernier, to 0.0001 in. It is provided with two milled heads having a spring ratchet between them which slips when contact is made. Some operators prefer to use the inside milled head, relying on their sensitiveness of touch to detect the instant of contact; others use the outside or ratchet head. The barrel of the instrument is covered with leather for mechanical and thermal protection.

Accompanying the instrument is a rectangular steel bar used as a reference or comparison bar (Fig. 2). It is provided with two center holes or gauge points 20 in. apart. Whenever a reading is taken on the member, a comparison reading is taken on the bar; the difference between these two readings is the measurement. This method of operation eliminates the effects of personal equation, as the same observer takes the readings on the bar and on the member. It also dispenses with the necessity of knowing the temperature of the instrument, as the unknown effect of this temperature is eliminated in subtracting the member reading from the bar reading.

FIG. 1.—HOWARD EXTENSOMETER.

FIG. 2.—COMPARISON BAR.

FIG. 3.—TRAMMEL BAR.

No expansion correction is necessary if the reference bar and the member have the same temperature; and, for this reason, the bar is allowed to rest on the member in order to equalize the temperatures before the extensometer is applied. Nevertheless, two thermometers are provided, one permanently inserted in the face of the reference bar and the other to be placed between the gauge points on the member being measured. When the measurement is taken, the two thermometer readings are observed and recorded, so that a correction may be made for any difference between them.

There is also furnished with the instrument a steel trammel bar (Fig. 3) holding two prick punches 20 in. apart. This is used for laying out the holes in the members to be tested.

The holes in the steel are drilled with a hand ratchet drill provided with combination bits, consisting of drill and countersink. The size of the drill is No. 57, and the countersink is 60 degrees. After drilling a hole, the edge between the countersink and cylindrical shaft is removed with a center punch which has been ground to an included angle of 55 degrees. With the tap of a hammer, the punch leaves a small conical seat to receive the measuring points of the extensometer which are also ground to 55 degrees. This special form of recessed gauge mark, with its combination of three surfaces, has a number of advantages: The extreme tips of the measuring points are not used, thus eliminating the errors which would result from the unavoidable wear of these tips; the contact is on an area, instead of a point or line, thereby reducing the wear of the measuring points and the errors from compression of the bearing surface; the bearing area is depressed below the surface of the steel, so that it is better protected from surface dirt and mechanical injury. The 60° countersink helps in guiding the measuring points to their seat. The cylindrical shaft will hold small particles of dust or grit remaining at the bottom of the hole without interfering with the precision of the measurement.

Oil, vaseline, or Ivory Black paint are used for protecting, filling, or covering the holes, depending on the interval between successive measurements. A pointed aluminum rod, a small cedarwood stick, and some absorbent cotton are used for cleaning the holes before taking any readings.

For the best operation of the instrument, two men are required: one to set the measuring points in the holes and hold the instrument square against the member; the other to operate the micrometer and take the readings.

The Howard strain gauge is probably the simplest and most accurate field instrument for measuring strains. It is free from the inertia and vibration errors which are inherent in automatic and recording types of extensometers; but it requires careful and experienced observers to attain the full degree of precision afforded by the instrument. Unless the readings are checked and all necessary precautions are observed, the apparent precision of the readings to 0.0001 in. will be illusory, and the measurements will be worthless.

Without mentioning all the precautions, most of which will readily suggest themselves to any one who has worked with instruments of precision, it may be stated in general that in order to get good results from the measurements, everything depends on the cleanliness of the holes, the proper placing of the strain gauge, the careful operation of the micrometer head, and due regard to temperature conditions.

Application to the Hell Gate Arch.

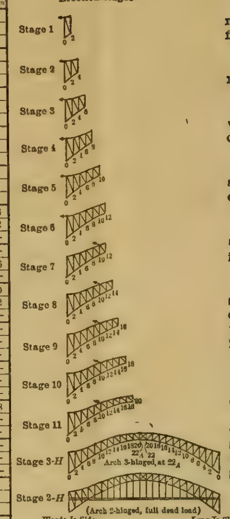
In the Hell Gate Arch, the lower chords present the most interesting problems in stress distribution. Moreover, they are the most important members, carrying nearly all the dead load and live load covering the whole span. For this reason, together with the restricted time afforded between erection stages to take more stress measurements, it was decided to confine the observations, in general, to the lower chord members.

As the half-arches were cantilevered out from shore, the successive erection stages were designated by numbers. The first six panels were built out with the lower stay acting, and the corresponding erection stages are numbered "One" to "Six", respectively. The transfer of the load to the upper stay produced the condition called Stage "Seven", and the erection of the seventh to the eleventh panels constituted Stages "Eight" to "Twelve", respectively. (Plate XXXVI.) The joining of the two half-arches at mid-span and the release of the back-stays brought the trusses to the three-hinged condition, which was designated as "Stage 3-*H*." Then followed the erection of the hangers, floor-beams, stringers, floor, and track, and the closing of

MEMBER		0–2				2–4				4–6				6–8				8–10				10–12			
		Primary Stress		Secondary Stress in % of Primary		Primary Stress		Secondary Stress in % of Primary		Primary Stress		Secondary Stress in % of Primary		Primary Stress		Secondary Stress in % of Primary		Primary Stress		Secondary Stress in % of Primary		Primary Stress		Secondary Stress in % of Primary	
Erection Stage	TRUSS	Calculated	Measured	Calculated Maximum	Measured Maximum	Calculated	Measured	Calculated Maximum	Measured Maximum	Calculated	Measured	Calculated Maximum	Measured Maximum	Calculated	Measured	Calculated Maximum	Measured Maximum	Calculated	Measured	Calculated Maximum	Measured Maximum	Calculated	Measured	Calculated Maximum	Measured Maximum
1	S. Truss, Long Is. Side	220	400	1260	122																				
	N. Truss, Long Is. Side	—	—																						
	N. Truss, Wards Is. Side	—	—																						
2	S. Truss, Long Is. Side	695	925	320	24	—	—	—	—																
	N. Truss, Long Is. Side	695	1125	320	90	—	—	—	—																
	N. Truss, Wards Is. Side	609	925	320	93	447	700	835	70																
3	S. Truss, Long Is. Side	—	—	—	—	828	610	225	184	—	—	—	—												
	N. Truss, Long Is. Side	—	—	—	—	828	860	225	214	—	—	—	—												
	N. Truss, Wards Is. Side	1660	1010	102	7	828	800	225	56	480	440	385	77												
4	S. Truss, Long Is. Side	2010	1890	58	10	1475	1590	85	[illegible]	800	890	101	97	232	475	235	118								
	N. Truss, Long Is. Side	2010	1540	[illegible]	134	1475	1475	85	66	800	1150	101	65	—	—	—	—								
	N. Truss, Wards Is. Side	2010	1860	[illegible]	10	1820	2010	85	46	1070	950	101	71	—	—	—	—								
5	S. Truss, Long Is. Side	3190	2925	28	26	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—				
	N. Truss, Long Is. Side	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—				
	N. Truss, Wards Is. Side	2660	2425	28	8	2220	2140	58	58	1410	1460	76	75	585	425	140	115	290	250	—	—				
6	S. Truss, Long Is. Side	—	—	—	—	3520	3465	30	11	2630	2550	51	28	1600	2210	105	44	910	1290	161	125	282	725	479	236
	N. Truss, Long Is. Side	3650	3110	6	68	3520	3300	30	34	2440	2590	51	54	1600	1375	105	67	900	1010	161	108	282	625	479	72
	N. Truss, Wards Is. Side	3580	3260	6	12	3237	3010	30	60	2354	2160	51	50	1379	1275	105	80	720	690	161	147	—	—	—	—
7	S. Truss, Long Is. Side	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—
	N. Truss, Long Is. Side	2080	—	—	—	1450	—	—	—	725	—	—	—	587	—	—	—	150	—	—	—	282	650	53	6
	N. Truss, Wards Is. Side	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—
8	S. Truss, Long Is. Side	2570	1525	95	37	2000	1890	104	58	1310	1175	102	96	830	1090	235	86	815	1090	300	131	940	1040	109	130
	N. Truss, Long Is. Side	2570	1540	95	81	2000	1910	104	110	1310	1075	102	45	830	440	235	136	815	1000	300	98	918	960	109	82
	N. Truss, Wards Is. Side	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—
9	S. Truss, Long Is. Side	—	—	—	—	2570	2375	74	51	—	—	—	—	—	—	—	—	1700	1550	132	78	—	—	—	—
	N. Truss, Long Is. Side	3060	1775	72	55	—	—	—	—	1920	1425	58	47	1560	1100	113	17	—	—	—	—	1390	—	—	—
	N. Truss, Wards Is. Side	31[illegible]	2675	72	30	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—
10	S. Truss, Long Is. Side	3640	2825	53	8	—	—	—	—	2820	2275	32	30	2410	2150	65	26	—	—	—	—	—	—	—	—
	N. Truss, Long Is. Side	—	—	—	—	3280	2640	53	63	—	—	—	—	—	—	—	—	2760	2860	106	70	3060	—	106	—
	N. Truss, Wards Is. Side	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—
11	S. Truss, Long Is. Side	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	4470	3990	97	38
	N. Truss, Long Is. Side	4300	—	—	—	4100	—	—	—	3720	—	—	—	3390	—	—	—	4000	—	—	—	—	—	—	—
	N. Truss, Wards Is. Side	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—
3H	S. Truss, Long Is. Side	7200	5910	12	—	7130	6850	9	—	6330	5775	9	—	5520	5610	11	—	5770	5790	12	—	5560	5610	18	—
	N. Truss, Long Is. Side	6310	5075	12	—	6050	5510	9	—	5520	5150	9	—	4730	4540	11	—	5160	5510	12	—	5150	5010	18	—
	N. Truss, Wards Is. Side	5000	5525	12	—	5800	5825	9	—	5300	5190	9	—	4650	4310	11	—	4920	4775	12	—	5080	5040	18	—
2H	S. Truss, Long Is. Side	—	—	—	—					—	—	—	—					—	—	—	—				
	N. Truss, Long Is. Side	11890	10150	2	23					11500	10990	5	13					11290	11225	4	13				
	N. Truss, Wards Is. Side	11890	11150	2	3					11500	11650	5	11					11290	10575	4	7				

MEMBER		12–14				14–16				16–18				18–20				20–22				$22–22_A$			
		Primary Stress		Secondary Stress in % of Primary		Primary Stress		Secondary Stress in % of Primary		Primary Stress		Secondary Stress in % of Primary		Primary Stress		Secondary Stress in % of Primary		Primary Stress		Secondary Stress in % of Primary		Primary Stress		Secondary Stress in % of Primary	
Erection Stage	TRUSS	Calculated	Measured	Calculated Maximum	Measured Maximum	Calculated	Measured	Calculated Maximum	Measured Maximum	Calculated	Measured	Calculated Maximum	Measured Maximum	Calculated	Measured	Calculated Maximum	Measured Maximum	Calculated	Measured	Calculated Maximum	Measured Maximum	Calculated	Measured	Calculated Maximum	Measured Maximum
8	S. Truss, Long Is. Side	310	690	339	152																				
	N. Truss, Long Is. Side	310	690	339	158																				
	N. Truss, Wards Is. Side	—	—	—	—																				
9	S. Truss, Long Is. Side	—	—	—	—	—	—	—	—																
	N. Truss, Long Is. Side	1030	—	—	—	338	—	—	—																
	N. Truss, Wards Is. Side	—	—	—	—	—	—	—	—																
10	S. Truss, Long Is. Side	—	—	—	—	—	—	—	—	—	—	—	—												
	N. Truss, Long Is. Side	2060	—	77	—	1130	—	83	—	375	—	165	—												
	N. Truss, Wards Is. Side	—	—	—	—	—	—	—	—	—	—	—	—												
11	S. Truss, Long Is. Side	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—								
	N. Truss, Long Is. Side	3310	—	—	—	2240	—	—	—	1250	—	—	—	405	—	—	—								
	N. Truss, Wards Is. Side	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—								
3H	S. Truss, Long Is. Side	5360	5560	24	—	5660	5770	26	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—
	N. Truss, Long Is. Side	5300	6050	24	—	5580	5990	26	—	6000	6590	32	—	6470	6910	37	—	6720	7210	38	—	7150	7076	48	—
	N. Truss, Wards Is. Side	5330	6090	24	—	5550	6060	26	—	6030	6260	32	—	6130	6760	34	—	6750	7200	48	—	—	—	—	—
2H	S. Truss, Long Is. Side	—	—	—	—					—	—	—	—					—	—	—	—	—	—	—	—
	N. Truss, Long Is. Side	11240	11610	6	19					11610	11725	7	15					11980	12125	11	16	11820	11775	6	27
	N. Truss, Wards Is. Side	11240	11015	6	6					11610	11315	7	7					—	—	—	—	—	—	—	—

Erection Stages



NOTES

All Primary Stresses are compression, and are given in pounds per square inch.

All Secondary Stresses are either tension or compression, representing equal values of opposite sign in the top and bottom fibers of a section.

All Secondary Stresses are given in percentages of the corresponding Primary Stresses.

Each Calculated Primary Stress is calculated for the actual weight and condition of structure and travelers at the time the corresponding stress was measured.

Each Measured Primary Stress is the average of twelve strain measurements, six points being observed at each end of each member.

Each Calculated Secondary Stress is the larger of the two secondary stresses calculated for the two ends of each member in each erection stage.

Each Measured Secondary Stress is the larger of the two secondary stresses found for the two ends of each member in each erection stage, the secondary stress at each end being obtained as one-half the difference between the average stresses in the top and bottom fibers of the cross-section.

For Stage 3-H, the Calculated Secondary Stresses were figured for the theoretical stage with no floor-beams or hangers erected.

As the field measurements were taken during the erection of floor-beams and hangers, with varying positions of the erection travelers, the Measured Secondary Stresses for this stage are not properly comparable with the Calculated Secondary Stresses and are therefore omitted from the tabulation.

EXTENSOMETER MEASUREMENTS
OF
STRESSES IN BOTTOM-CHORD
MEMBERS OF HELL GATE
ARCH BRIDGE.

the crown hinge, bringing the arch to the final or two-hinged condition known as "Stage 2-*H*."

To secure full information as to the distribution of stress in any member, it is necessary to take measurements at not less than three, and preferably four, points of the cross-section near each end of the member. The intensity of stress at any other point of the member can then be found by planar and linear interpolation.

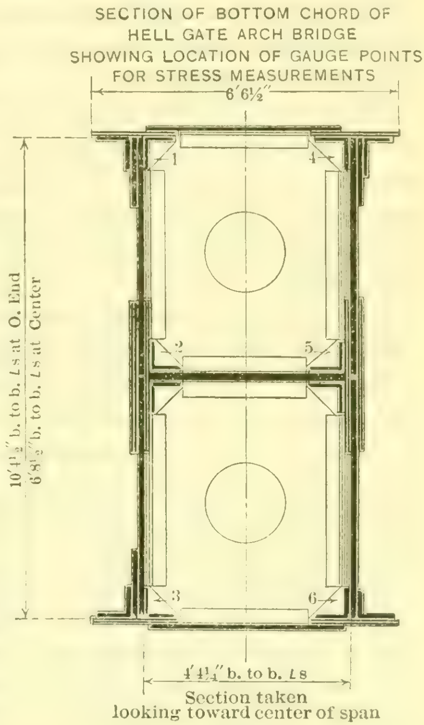


Fig. 4.

The lower chords of the Hell Gate Arch have a double rectangular section (Fig. 4), consisting of two compartments separated by a horizontal diaphragm. On account of the mass of metal concentrated at and near this diaphragm, measurements at the four extreme corners of the section would not be fairly representative of the conditions throughout the entire area. Any difference of temperature between the horizontal diaphragm and the outside webs tends to produce internal stresses in the member, resulting in a difference between the stress in the diaphragm and the average stress throughout the rest of the cross-section. Furthermore, the beveling of the outer thirds of the webs (Fig. 6), produces a concentration of stress in the middle third of the joint. For these reasons it was judged necessary to take six readings at each cross-section, instead of four; the two additional readings being taken at mid-height in the vertical webs, as indicated in Fig. 4.

The large dimensions of the chord sections permitted all the measurements to be taken on the inside of the members. This afforded better protection for the gauge points from rain and dirt.

Incidentally, it secured better protection for the observers from falling drift-pins and rivets.

The gauge points were drilled and the initial readings taken while the members were still on the dock or on the car-floats. The points were located in the vertical legs of the 8 by 8-in. flange angles connecting the vertical and horizontal webs, and as near the panel points as possible, but not within gusset or splice-plates. There were thus twelve gauge points in each member, each gauge point consisting of two holes 20 in. apart. The exact location of each gauge point was noted for permanent record.

For designation, the gauge points were numbered from 1 to 6 at each section, and to these figures was prefixed the number of the member. Thus, "hole 681" denotes the upper, left-hand gauge point near panel point 6 of member 6-8; similarly, "hole 864" denotes the upper right-hand gauge point (No. 4) near panel point 8 of the same member.

The number of the hole and that of the erection stage thus suffice to identify any extensometer reading.

Because of the more rapid erection on the Wards Island side, one half-truss on that side had to be omitted from the programme of measurements. The remaining three sets of measurements, however, are sufficient to furnish information as to any variations from the symmetrical disposition of stresses about the longitudinal and transverse center-lines of the bridge.

There were thus twelve pairs of holes in each of thirty-five chord members to be drilled; and stress readings were to be taken at these points in fourteen different stages. This meant a total of more than 3 000 extensometer measurements in the regular schedule, besides special measurements in the back-stays and the eye-bar connection links.

In some of the later stages, time did not permit all the chord stresses to be measured; and, therefore, alternate members were omitted.

On account of the difficulty of traveling through the members with the instruments, involving crawling through numerous diaphragm holes, much time was consumed in going to and from the points of measurement. Consequently, it was seldom possible to measure more than two members in a day.

TABLE 1.—TYPICAL FIELD RECORD SHEET.
RECORD OF EXTENSOMETER READINGS AT HELL GATE ARCH BRIDGE.

Location: Long Island End, Member 0-2, South Truss, and Member 2-4, North Truss.

Date: Sept. 15, 1915. Time: 8.30 A. M. to 4.15 P. M. Weather: Clear.
Observers: O. H. S. Koch and A. V. O'Donnell.

No.	Temperature. Degrees Fahr.			Reading.		Difference in Length Between Bar and Member.		Length corrected to 60° Fahr.	Location on member where measurements were taken.
	Air.	Bar.	Member.	Bar.	Member.	Measured.	Corrected for temp.		
				South Truss, Member 0-2.					
2025	......	78.5	78	9983	9877	—0106	—0105	19.9895	025
2026	......	80	78	9978	9850	—0128	—0125	19.9875	024
2027	......	80	78.5	9977	9855	—0122	—0120	19.9880	021
2028	......	80	79.5	9977	0139	+0162	+0163	20.0163	022
2029	......	81	78	9978	9898	—0080	—0076	19.9924	205
2030	......	81	79	9978	9935	—0043	—0040	19.9960	204
2031	......	82	80	9978	9739	—0239	—0236	19.9764	201
2032	......	82	80	9978	9878	—0100	—0097	19.9903	202
2033	......	83	81	9981	9896	—0085	—0082	19.9918	023
2034	......	83	79.5	9980	9966	—0014	—0009	19.9991	026
2035	......	83	80	9980	9892	—0088	—0084	19.9916	206
2036	84	83.5	84	9980	9930	—0050	—0051	19.9949	203
				North Truss, Member 2-4.					
2037	......	85.5	83	9982	9952	—0030	—0027	19.9973	245
2038	......	85.5	87	9982	9975	—0007	—0009	19.9991	244
2039	......	86	86	9982	9995	+0013	+0013	20.0013	242
2040	......	86	88	9981	9950	—0031	—0034	19.9966	241
2041	......	86.5	85.5	9982	9990	+0008	+0009	20.0009	422
2042	......	86.5	89	9981	9919	—0062	—0065	19.9935	421
2043	......	87	83.5	9981	0013	+0032	+0037	20.0037	425
2044	......	87	87.5	9981	0340	+0359	+0358	20.0358	424
2045	......	87	86	9983	9991	+0008	+0009	20.0009	246
2046	......	87	90.5	9982	0045	+0063	+0058	20.0058	243
2047	......	87.5	87	9981	9974	—0007	—0006	19.9994	423
2048	86	87.5	86	9981	0008	+0027	+0029	20.0029	426

NOTES.—No. = serial number of observation. Under temperature: air = atmospheric temp., bar = temp. of std. reference bar, member = temp. of member or metal. Under reading: bar = strain gauge measurement of std. reference bar, member = strain gauge measurement of member or metal.

REMARKS.—Erection Stage No. 10, traveler still at 15–17. Wind: southwest, mild.

Signed, OTTO H. S. KOCH.

Table 1 shows a typical field-office record sheet. It also serves to indicate the arrangement in the field notebook, as the record sheet is practically a typewritten copy of the field entries.

From the record sheets the measurements were entered and computed on office cards, illustrated by Tables 2, 3, and 4. One of these cards was provided for each member, with spaces for entering the

observed strains in each erection stage. Following the numbers or marks of the gauge points there is a column headed "Initial" in which are recorded the lengths between gauge points before the member is stressed. The standard length of 20 in. is understood to be added to each of these figures. In the remaining columns the figures denote the actual strain, that is the increase or decrease from the initial gauge length for the different erection stages. Each unit of strain (0.0001 in.) represents a fiber stress of 150 lb. per sq. in.; hence the sum of the six strains at a section, multiplied by 25, gives the average unit stress in the member. The calculated stresses, inserted for comparison with the measured stresses, were computed from the shipping weights of the erected members, with an allowance for erection material.

The extreme strain readings observed for each member, multiplied by 150, give the minimum and maximum fiber stresses in the member.

From the cards the summary table (Plate XXXVI) was compiled, giving a comparison of the calculated and the observed stresses in the various erection stages.

The secondary stresses were computed from the strain measurements, as follows: The average of the observed strains in Holes 1 and 4 at any section gave the upper fiber stress; the average for Holes 3 and 6 gave the bottom fiber stress; one-half the difference between these two extreme fiber stresses gave the secondary stress. This result divided by the average stress in the member gave the secondary stress as a percentage of the primary stress.

Results of the Stress Measurements.

A tabulation of the results of the stress measurements on the Hell Gate Arch during erection is shown on Plate XXXVI. The notes and data on that plate render it self-explanatory.

The results of the observations have been studied from various angles, in order to determine the following relations:

1.—Comparison of the measured average stresses at the two ends of each member, as an index of the precision of the method;

2.—Comparison of measured with calculated primary stresses;

3.—Comparison of extreme secondary stresses with primary stresses, in order to establish empirical relations between the two;

TABLE 2.—Typical Office Record Card.

Extensometer Measurements of Erection Stresses, Hell Gate Arch.

Wards Island End. North Truss. Member 0-2.

	Hole.	Initial.	Stage 1.	Stage 2.	Stage 3	Stage 4.	Stage 5.	Stage 6.	Stage 7.	Stage 8.	Stage 9.	Stage 10.	Stage 11.	3-*H*.	2-*H*.
Lower end.	021	— 0020	....	— 10	— 12	— 16	— 23	— 26	....	....	— 24	....	....	— 39	— 77
	022	+ 0013	....	0	— 7	— 13	15	— 17	....	....	— 16	....	....	— 34	— 83
	023	— 0007	....	+ 1	— 12	— 13	— 20	— 27	....	....	— 13	....	....	— 36	— 73
	024	+ 0011	....	— 13	— 5	— 12	16	— 19	....	....	— 23	....	....	— 38	— 60
	025	+ 0048	....	— 12	— 3	— 15	14	16	....	....	— 19	....	....	— 38	— 76
	026	0031	....	1	— 4	— 12	— 14	24	....	....	— 17	....	....	— 33	— 67
	Total.....	....	....	35	— 43	— 81	102	129	....	....	— 112	....	....	— 218	— 436
	Aver. Stress..	....	....	— 875	— 1 075	2 025	— 2 550	— 3 225	....	....	2 800	....	....	— 5 450	— 10 900
Upper end.	201	+ 0002	....	— 3	— 7	— 9	18	— 15	....	....	— 13	....	....	— 29	— 61
	202	— 0001	....	— 11	— 13	— 13	22	— 25	....	....	— 17	....	....	— 35	— 96
	203	+ 0002	....	+ 2	— 7	— 11	17	24	....	....	— 8	....	....	— 34	— 75
	204	+ 0008	....	18	— 6	— 16	14	— 23	....	....	— 31	....	....	— 49	— 72
	205	+ 0025	....	5	— 1	— 10	10	— 21	....	....	18	....	....	— 40	— 108
	206	+ 0011	....	4	— 6	— 9	11	24	....	....	15	....	....	— 37	— 68
	Total....	....	....	39	— 40	— 68	92	132	....	....	102	....	....	— 224	— 480
	Aver. stress..	....	....	975	1 000	— 1 700	— 2 300	3 300	....	....	— 2 550	....	....	5 600	— 12 000
	Calc. stress..	....	....	669	1 660	— 2 010	— 2 660	3 580	....	....	3 110	....	....	5 900	11 890
	Min. stress...	....	....	+ 300	— 150	— 1 350	— 1 500	2 250	....	....	1 200	....	....	4 350	— 9 000
	Max. stress..	....	....	2 700	— 1 950	— 2 400	— 3 450	4 050	....	....	— 4 650	....	....	7 350	— 16 200

TABLE 3.—TYPICAL OFFICE RECORD CARD.

EXTENSOMETER MEASUREMENTS OF ERECTION STRESSES, HELL GATE ARCH.

Long Island End. North Truss. Member 2-4.

	Hole.	Initial.	Stage 1.	Stage 2.	Stage 3.	Stage 4.	Stage 5.	Stage 6.	Stage 7.	Stage 8.	Stage 9.	Stage 10.	Stage 11.	3-*H*.	2-*H*.
Lower end.	241	— 0017	..	..	— 3	— 5	..	— 9	..	— 8	..	— 17	..	— 32	....
	242	+ 0024	..	..	— 4	— 2	..	— 16	..	— 5	..	— 11	..	— 31	....
	243	+ 0075	..	..	— 2	— 6	..	— 27	..	— 10	..	— 17	..	— 34	....
	244	+ 0015	..	..	— 12	— 12	..	— 22	..	— 22	..	— 24	..	— 38	....
	245	— 0012	..	..	— 9	— 14	..	— 22	..	— 11	..	— 15	..	— 36	....
	246	+ 0032	..	..	— 10	— 25		— 34	..	— 16	..	— 23	..	— 47	....
Total......		....	..	..	— 40	— 64	..	— 130	..	— 72	..	— 107	..	— 218	....
Aver. stress...		....	..	..	— 1 000	— 1 600	..	— 3 250	..	— 1 800	..	— 2 675	..	— 5 450	....
Upper end.	421	— 0036	..	..	— 18	— 11	..	— 28	..	— 28	..	— 29	..	— 44	....
	422	+ 0023	..	..	+ 1	— 4	..	— 18	..	— 7	..	— 14	..	— 33	....
	423	— 0001	..	..	+ 12	— 1	..	— 16	..	— 2	..	— 5	..	— 28	....
	424	+ 0386	..	..	— 16	— 18	..	— 29	..	— 31	..	— 28	..	— 45	....
	425	+ 0057	..	..	— 12	— 18	..	— 23	..	— 12	..	— 20	..	— 42	....
	426	+ 0037	..	..	+ 4	— 2	..	— 20	..	— 1	..	— 8	..	— 33	....
Total......		....	..	..	— 29	— 54	..	— 134	..	— 81	..	— 104	..	— 225	....
Aver. stress...		....	..	..	— 725	—1 350	..	— 3 350	..	— 2 025	..	— 2 600	..	— 5 625	....
Calc. stress...		....	..	..	— 828	—1 475	..	— 3 520	..	— 2 000	..	— 3 280	..	— 6 050	....
Min. stress....		....	..	..	+1 800	— 150	..	— 1 350	..	— 150	..	— 750	..	— 4 200	....
Max. stress...		....	..	..	—2 700	—3 750	..	— 5 100	..	— 4 650	..	— 4 350	..	— 7 050	....

4.—Comparison of calculated with measured secondary stresses, in order to ascertain the effects of special features of fabrication and erection on these stresses; and

5.—Comparison of calculated with measured extreme fiber stresses, in order to determine the resultant effect of all variations in primary and secondary stress.

COMPARISON OF MEASUREMENTS AT THE TWO ENDS OF EACH MEMBER.

By averaging the six measurements near each end of a member, two values are obtained for the average intensity of stress in the member. The difference between these two values, provided there is

TABLE 4.—TYPICAL OFFICE RECORD CARD.
EXTENSOMETER MEASUREMENTS OF ERECTION STRESSES, HELL GATE ARCH.
Wards Island End. North Truss. Member 4-6.

	Hole.	Initial.	Stage 1.	Stage 2.	Stage 3.	Stage 4.	Stage 5.	Stage 6.	Stage 7.	Stage 8.	Stage 9	Stage 10.	Stage 11.	*3-H.*	*2-H.*
Lower end.	461	— 0078	..	..	— 12	— 11	— 17	— 15	..	..	..	..	..	— 37	— 62
	462	+ 0015	..	..	— 7	— 6	— 17	— 13	..	..	..	..	..	— 35	— 75
	463	— 0037	..	..	— 14	— 12	— 14	— 27	..	..	..	..	..	— 43	— 76
	464	— 0016	..	..	+ 9	+ 4	+ 5	— 1	..	..	..	..	..	— 20	— 51
	465	— 0024	..	..	+ 4	— 5	— 11	— 14	..	..	..	..	..	— 35	— 76
	466	— 0068	..	..	+ 2	— 9	— 5	— 18	..	..	..	..	..	— 38	— 70
	Total	....	..	..	— 18	— 39	— 59	— 88	..	..	..	..	..	— 208	— 410
	Aver. stress	....	..	..	— 450	— 975	— 1 475	— 2 200	..	..	..	..	..	— 5 200	— 10 250
Upper end.	641	— 0032	..	..	— 4	— 14	— 21	— 17	..	..	..	..	..	— 44	— 70
	642	— 0012	..	..	— 5	— 8	— 14	— 17	..	..	..	..	..	— 42	— 90
	643	+ 0023	..	..	— 5	— 3	— 1	— 15	..	..	..	..	..	— 32	— 69
	644	— 0004	..	..	— 2	— 4	— 10	— 8	..	..	..	..	..	— 28	— 81
	645	— 0367	..	..	— 1	— 11	— 11	— 15	..	..	..	..	..	— 34	— 139
	646	+ 0040	..	..	0	+ 3	— 1	— 13	..	..	..	..	..	— 27	— 73
	Total	....	..	..	— 17	— 37	— 58	— 85	..	..	..	..	..	— 207	— 522
	Aver stress	....	..	..	— 425	— 925	— 1 450	— 2 125	..	..	..	..	..	— 5 175	— 13 050
	Calc. stress	....	..	..	— 480	— 1 070	— 1 440	— 2 354	..	..	..	..	..	— 5 300	— 11 500
	Min. stress	....	..	..	+ 1 350	+ 600	+ 750	— 150	..	..	..	..	..	— 3 000	— 7 650
	Max. stress	....	..	..	— 2 100	— 2 100	— 3 150	— 4 050	..	..	..	..	..	— 6 600	— 20 850

no inequality of respective cross-sections, is an index of the precision of the observations. The average value of this difference for all the lower chord measurements during erection was 140, and the greatest value was 500 lb. per sq. in. Consequently, by the theory of errors, the result obtained by averaging the two end stresses has an average probable error of 47 and a maximum probable error of 170 lb. per sq. in.

The foregoing figures represent the combined effect of all personal and instrumental errors in taking the observations. A very small error in each of the instrumental readings will account for these results. Thus, an inaccuracy of 0.0002 in. in each micrometer reading,

and of 1° Fahr. in each temperature reading, will produce the foregoing average probable error, even if all inaccuracies compensate according to the theory of probabilities. A very small fraction of the same inaccuracies will suffice to account for the observed discrepancies, if the inaccuracies do not compensate fully according to the probability theory. As the least reading of the micrometer is 0.0001 and of the thermometer is 1° Fahr., the results given indicate that the work, as a whole, was carefully executed, and that the utmost precision afforded by the method was actually attained.

Another fact to be observed is that the magnitude of the differences between the end measurements has no connection with the magnitude of the respective stresses. This method of measuring the stresses makes the errors independent of the intensity of the stress. No matter how small the stress, the result, as just shown, may be in error by as much as 170 lb. per sq. in. These considerations indicate the unsuitableness of this method of stress measurement for the smaller stresses, as the results would be too erratic. Although the stresses measured on the Hell Gate Bridge ranged from 400 to 20 000, the writer would not recommend the use of the extensometer for stresses of less than 1 000 lb. per sq. in.

The foregoing discussion of results does not include the final stage (2-*H*). In that stage, in the members near the ends of the span, the upper end in every case was found to present a larger average stress than did the lower end. The differences ranged from 450 to 2 800 lb. per sq. in. This anomalous result appears to arise from some unexplained disturbance of stress distribution, and does not represent any error of observation.

Comparison of Measured with Calculated Stresses.

A comparison of measured with calculated stresses discloses a greater discrepancy than is found between the measurements at the two ends of a member. This is due to other factors, besides instrumental and personal inaccuracies, entering into consideration.

The results of the comparison, including all measurements on chords, eye-bars, and back-stays, are as follows:

For stresses in the ranges 0-1 000, 1 000-3 000, 3 000-5 000, 5 000-10 000, and 10 000-20 000, the average percentage discrepancy between the calculated and measured stresses was 50, 17, 13, 8, and 6%, respec-

tively. These percentages indicate that the method is of little value for checking calculated stresses of less than 1 000 lb. per sq. in., but is of increasing value as the intensity of stress increases.

The average discrepancies in the foregoing ranges were 223, 313, 498, 525, and 829 lb. per sq. in., respectively, and may be expressed very closely by the formula,

$$\left.\begin{array}{r}\text{Average discrepancy between}\\ \text{calculated and measured stress}\end{array}\right\} = 5\% \text{ of the stress} + 200.$$

The greatest discrepancies in the foregoing ranges were 535, 1 045, 1 285, 1 290, and 1 740, respectively, and may be expressed approximately by the formula,

$$\left.\begin{array}{r}\text{Maximum discrepancy between}\\ \text{calculated and measured stress}\end{array}\right\} = 5\% \text{ of the stress} + 1\,000.$$

It will be observed that these formulas contain both a percentage factor and an absolute term, indicating that the discrepancy between calculation and measurement is the resultant of two groups of errors, one dependent on, and the other independent of, the magnitude of the stress.

The absolute term in these formulas (average 200, maximum 1 000 lb. per sq. in.) may be assumed to represent the actual inaccuracy of the measurements, or the total effect of personal, instrumental, and physical errors.

The most important of the physical errors are those due to thermal effects. An error of 1° Fahr. in determining the temperature of the steel at the point of measurement, or a change of 1° Fahr. in the temperature of the instrument during any observation, produces an error of 195 lb. per sq. in. in the resulting stress. The first error named is not necessarily due to incorrect reading of the thermometer, but may be caused by a difference in temperature between the inside and the outside of the thick webs composing the members. Such errors, as a rule, are not compensating; they would naturally occur in the same direction at both ends of a member, and, therefore, would not appear in the difference between the two end measurements.

Other errors arise from differences in temperature between different points of the large chord sections. Thus, on a sunny day, the outside webs and covers may be hotter than the middle diaphragm. The resulting differences of expansion produce a redistribution of

the stress in the cross-section. Although the actual average stress throughout the entire section must remain unchanged, the measured average stress will be affected by these internal temperature strains, as the readings are taken at a limited number of points in the cross-section.

It is difficult to estimate the total probable error due to these effects of temperature. Nevertheless, from a study of the results of the measurements, it appears that the effect of the combined temperature errors, together with any other unascertainable physical effects, is to increase the probable error of the stresses, as previously determined, from 47 (average) and 170 (maximum) to 200 (average) and 1 000 (maximum).

The foregoing discussion should serve to emphasize the importance of careful attention to temperature conditions in taking observations; and it explains why cloudy days should be selected in order to obtain the most accurate results.

After deducting all personal, instrumental, and temperature errors, represented by the absolute term in these formulas, there still remains a discrepancy between the calculated and the measured stresses represented by the percentage term (5%). This covers all percentage errors in the measured and the calculated stresses, that is, all errors which are proportional to the magnitude of the stress.

In the measured stresses, the only percentage error arises from variations in the value of the modulus of elasticity from the assumed value of 30 000 000. Such variation does not affect the accuracy of the measurements, but simply the conversion of the measured strains into intensities of stress. The resulting stress is then affected by a percentage error equal to the percentage variation in the value of E, and this may amount to $\pm$ 3 per cent.

In the calculated stresses, all errors are percentage errors. They are caused principally by variations in the loading and cross-sections from the values assumed. In the assumed loads there is a probable error of about $\pm$ 3%, on account of the uncertainty in allowing for the weight of traveler, track, rivets, pins, staging, etc. In the cross-sections there is a probable error of $\pm$ 2% on account of the difference between actual and theoretical sections.

The combined effect of the foregoing errors in assuming the loading, cross-sections, and value of E, will account for the percentage

term (± 5%) in the discrepancy between calculated and measured stresses.

The major part of this discrepancy of 5% arises from the errors in the calculated stress. This demonstrates the futility of excessive refinement in the calculation of stresses.

Comparison of Extreme Secondary Stresses with Primary Stresses.

The object of this comparison was to establish a relation between the average stresses and the extreme variations from these average stresses in the members of the structure. The information derived from such comparisons should be of value in guiding the selection and specification of working stresses for bridge materials. For this purpose it appeared desirable to get the extreme variation of fiber stress in each member, without regard to the cause or causes producing it. Such secondary stresses would include, not only the stress from bending of the members in the plane of the truss, as usually computed, but also any horizontal or lateral bending stresses from wind and transverse strains, effect of shop inaccuracies, internal temperature strains, lack of uniformity of material, and any other possible causes.

As a fair measure of this extreme secondary stress, including the resultant effect of all possible contributing elements, the difference was taken between the average of the twelve measurements in a member and that one of the twelve measurements departing most widely from the average.

A comparison of the secondary stresses thus obtained with the corresponding primary (or average) stresses yielded the following results:

For primary stresses in the ranges 0-1 000, 1 000-2 000, 2 000-3 000, 3 000-5 000, 5 000-7 000, and 7 000-8 000, the percentages of extreme secondary stress averaged 268, 148, 71, 59, 37, and 32, respectively. This illustrates the diminishing relative importance of secondary stresses with increasing primary stress.

Although the percentages diminish in the foregoing series of ranges, the absolute amounts of the secondary stress show a small increase, averaging 1 580 lb. per sq. in. in the lowest range and 2 370 lb. per sq. in. in the highest range.

In the one hundred cases represented in the foregoing results, the greatest extreme secondary stress that appears is 3 700 lb. per sq. in.

Plotting the results with primary and extreme secondary stresses, respectively, as co-ordinates, nearly all the observations were found to be included in a belt between two lines having a 12% slope. From this may be deduced the relation,

$$\left.\begin{array}{l}\text{Extreme secondary}\\ \text{stress in a member}\end{array}\right\} = 12\% \text{ of primary stress} + K,$$

where K varies from 600 to 2 600, with an average value of 1 600 lb. per sq. in.

The form of this expression, a percentage term plus an absolute term, indicates that the measured secondary stresses include effects proportional to the direct stress in the member as well as effects independent of that stress. The latter effects, constituting the major portion of the measured secondary stresses, are produced by such causes as the bending of a member due to its own weight, wind and lateral strains, internal temperature strains, erection strains, etc.

The proportional component or percentage term in the formula represents the secondary stresses resulting from the direct primary strains. It amounts to only 12% of the primary stresses in the case of the Hell Gate chords. It is interesting to note that this component of the secondary stress is smaller than the combined effect of the other contributions which are generally omitted from consideration in the computation of secondary stresses.

Combining the results of this and the preceding comparison, we may establish certain conclusions for guidance in specifying extreme working stresses for bridge members. As the maximum discrepancy between calculated and measured stress is 5% — 1 000, and as a possible extreme value of the secondary stress is 12% + 2 600, it appears that the sum of these two variations should be left as a necessary margin between the elastic limit of the material and the maximum calculated primary stress. As the Hell Gate chords had certain special features tending to reduce the secondary stresses, it is probable that the 12% factor is not typical, and that about 20% would be a better allowance for most bridges. Hence, about 25% + 4 000 should be deducted from the minimum elastic limit of the material in order to obtain the safe working stress for bridge members.

It is recognized, of course, that a rule of this form can properly apply only to structures in which the secondary stresses have normal

or average values. Where unusually large secondaries may be anticipated, these should receive special investigation and attention.

COMPARISON BETWEEN CALCULATED AND MEASURED SECONDARY STRESSES.

In order to be comparable with the calculated secondary stresses due to bending of the members in the plane of the truss, the "measured secondary stresses" had to be defined as one-half the difference between the average measured stresses in the top and bottom fibers of the cross-section. Any lateral variation in the stress had to be ignored.

The "calculated secondary stresses" were obtained by analytical methods for Stages 2, 4, 6, 8, 10, 3-*H*, and 2-*H*, and by interpolation for the intermediate stages.

Both the measured and the calculated values were reduced to percentages of the corresponding primary stresses and recorded in the table, Plate XXXVI.

It should be noted that the high percentages always occur with low primary stresses. The law of variation is asymptotic.

The average of the calculated secondary stresses, for all bottom chord members and all erection stages, was about 1 050 lb. per sq. in. The average of all measured secondary stresses was only 700 lb. per sq. in.

The highest calculated secondary stress in any stage was 2 920; and the corresponding measured stress was 1 990 lb. per sq. in.

A comparison of individual values of calculated and of measured secondary stresses is not very illuminating, as any systematic variations are more or less obscured by the effects of erratic readings and other disturbing factors.

More instructive results are obtained by averaging the individual readings in groups so as to eliminate accidental variations and disclose the true relations between the calculated and the measured values.

If the secondary stresses are grouped and averaged in ranges of percentages, the following comparison is obtained:

In the ranges,

0-20%, 20-40%, 40-100%, 100-300%, and more than 300%,

the average calculated secondary stresses were:

6%, 30%, 69%, 143%, and 473%,

and the corresponding average measured secondary stresses were:

26%, 28%, 46%, 93%, and 110%, respectively.

From these figures, the following relations are evident:

1.—Except for the smaller percentage of secondary stresses, the measured values are consistently lower than the calculated values.

2.—As the percentage of calculated stresses increases, the percentage of measured values also increases, but not as rapidly.

3.—For the highest percentages of secondary stresses, the measured values amount to only a small fraction of the calculated values.

The lower values of the measured secondary stresses are partly due, in the case of the Hell Gate Arch, to the three-faced joints in the bottom chords; but, to a large extent, the effect is to be ascribed to a readjustment of strains tending to relieve the higher secondary stresses. As this effect obtains in all structures, it may be concluded that the actual secondary stresses are generally lower than the calculated values.

These results may be expressed by an approximate empirical relation between the calculated and the measured percentages of secondary stress. If the various observations are plotted with these percentages as co-ordinates, all but the extreme points are found to be grouped along a straight line of the formula,

$$\text{Percentage of measured secondary stress}$$
$$= \frac{1}{2}\,(\text{percentage of calculated secondary stress}) + 15.$$

This expression may be interpreted as follows:

1.—The factor, $\frac{1}{2}$, was the average ratio of actual to calculated stresses, and represents the reduction of the bending strains by the yielding of the joints and the internal readjustment of the structure to relieve the secondary stresses. This factor would probably have a higher value in other structures.

2.—The absolute term, 15%, represents additional secondary strains due to factors not included in the computations, such as inaccuracies of fabrication, effects of temperature, dead weight of members, eccentric bearings, etc. This term would probably have a lower value in other structures.

For the most instructive comparison, it is necessary to average the calculated and measured values in groups corresponding to the successive erection stages. This yields the results shown in Table 5.

TABLE 5.

Stage.	Average primary stress.	Average Secondary Stress.	
		Calculated.	Measured.
2	612	1 220	485
4	1 370	703	547
6	2 040	847	796
8	1 254	1 335	711
10	2 393	1 629	725
3-*H*	5 820	557	832
2-*H*	11 620	513	1 140 (296)

The values in Table 5 are plotted in graphs in Fig. 5. Three curves are shown: primary stresses, calculated secondary stresses, and measured secondary stresses.

The curve of primary stresses shows a large drop from Stage 6 to Stage 8, representing the reduction in stresses when the load was transferred from the lower to the upper back-stay.

The curve of calculated secondary stresses is rather high in the first two stages. This is simply because at these stages only one or two panels were erected, and these end panels usually have higher secondary stresses than the intermediate panels of the span. From Stage 4 to Stage 12, the calculated secondary stresses increase continuously with the increasing deflections. At Stage 3-*H* there is a sudden drop, as the 3-hinged condition has very small deflections and, consequently, low secondary stresses.

The measured secondary stresses for the first few stages are very considerably below the calculated values. This proves that the three-faced joints (Fig. 6) acted partly as hinges which relieved the secondary stresses. As the direct stress increased, however, the joints were compressed, the bearing area was enlarged, and the drift-pins in the outer thirds of the joint began to take stress. This accounts for the increasing slope of the curve as Stage 6 is approached, and the diminishing difference between calculated and measured values. From Stage 6 to Stage 8 there is a decline in the measured secondary stresses due to the temporary release in the direct compression at the joints, combined with a reversal of flexure at some of the joints during this transition. Beyond Stage 8 the measured secondary stresses increase continuously.

The final drop in the calculated stress curve is not duplicated in the curve of measured secondary stresses. In the former curve, the

strains for Stages 12 and 3-*H* are the results of independent computations for two entirely distinct erection conditions, and this accounts for the break in the curve. In the measured secondary stresses, how-

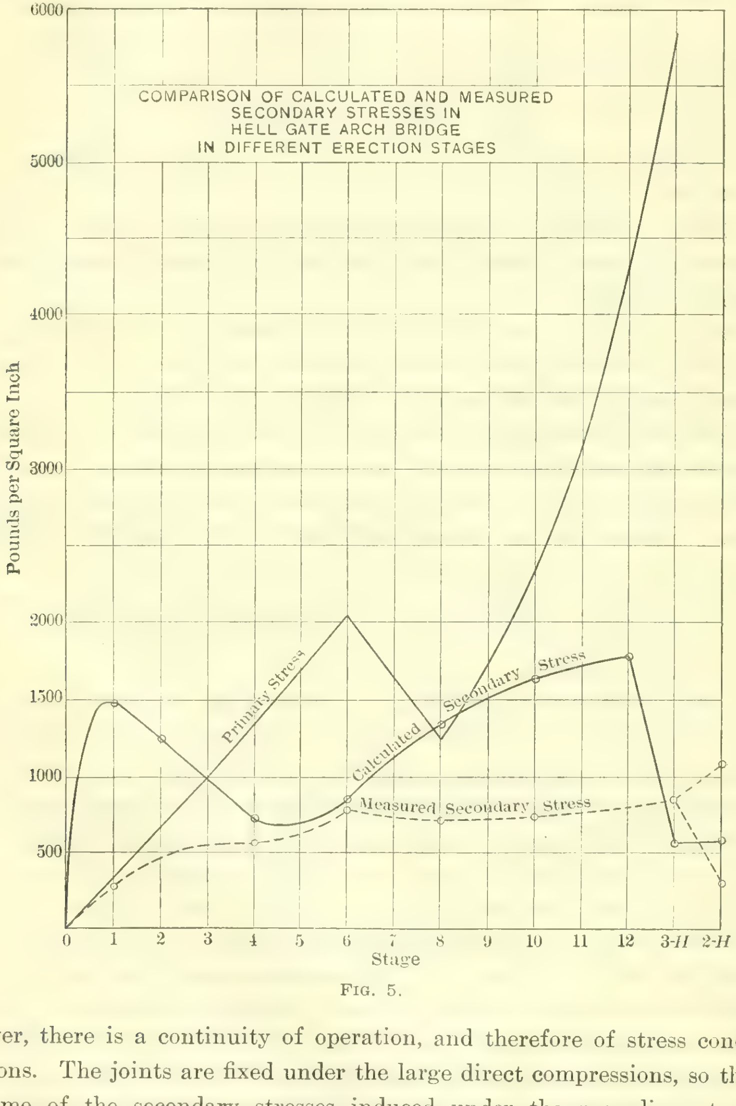


FIG. 5.

ever, there is a continuity of operation, and therefore of stress conditions. The joints are fixed under the large direct compressions, so that some of the secondary stresses induced under the preceding stages persist in the structure when Stages 3-*H* and 2-*H* are reached.

The stresses in Stage 3-*H* were measured before the joints were riveted. During the operation of riveting, as the drift-pins, one by one, were replaced by rivets, there was a gradual redistribution of stresses at each joint, consisting in a transfer of pressure from the outer fibers to the middle of the section. There was no perceptible rotation of the joints, however, as their compression had reached a stage which precluded the possibility of any hinge action. Consequently, the measured secondary stresses, defined as one-half the difference between the top and bottom fiber stresses, showed no drop between Stages 3-*H* and 2-*H*. (See upper branch of graph.)

If, however, the secondary stresses are defined as the excess compression in the top or the bottom fibers over the average stress in the section, the measured secondary stresses in the final stage present a distinct reduction from the preceding stage as a result of the redistribution of stress to relieve the outer fibers. The measured secondary stresses for Stage 2-*H*, thus computed, have an average value of only 296, as compared with the average calculated value of 513. (See lower branch of graph.)

On the whole, the curves (Fig. 5) show that the measured secondary stresses are lower than the calculated values, the average ratio between them being somewhat more than one-half.

Comparison of Calculated with Measured Extreme Fiber Stress.

In the final stage (2-*H*), the extreme fiber stresses, representing the maximum combination of primary and secondary stresses, yielded the following comparison:

Member:	0–2	4–6	8–10	12–14	16–18	20–22
Calculated:	12 160	12 060	11 770	11 950	12 370	13 270
Measured:	11 300	11 650	11 450	11 800	11 950	13 300

This striking comparison shows that, in the Hell Gate Arch chord members, the factors tending to increase the calculated extreme fiber stresses and those tending to reduce them very nearly balance each other. Were it not for the three-faced butt joints and the deferred riveting, the measured extreme stresses would have exceeded the calculated values.

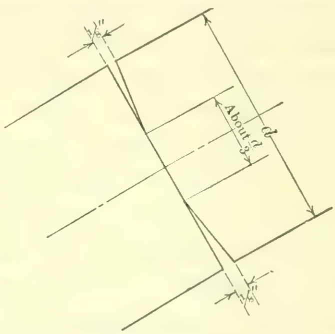
Other Applications of the Extensometer.

In order to observe the movements at the lower chord splices, the extensometer was applied across the opening at some of the panel points in various erection stages.

The initial value of this opening, at the top and bottom of each butt joint, was $\frac{1}{8}$ in. (Fig. 6).

To illustrate the character of the observations, the results for panel point 2 of the Long Island side will be summarized.

Readings were taken at each corner of the joint in both trusses. Averaging the four top measurements and the four bottom measurements, respectively, and comparing each observation with the initial measurement, the following results were obtained:



Joint of Bottom Chord
FIG. 6.

Stage.	4.	6.	8.	10.	3-*H*.	2-*H*.
Top.	Initial	— 56	—67	— 95	—198	—1 186
Bottom.	Initial	—140	—45	—100	—226	—1 050

It is seen from these figures that, between Stages 4 and 6, the top of the joint closed 0.0056 in. and the bottom closed 0.0140 in., representing a negative rotation.

Between Stages 6 and 8, the top of the joint closed 0.0011 in. and the bottom *opened* 0.0095 in., representing a positive rotation. This corresponds to the reversed flexure at this panel point when the load was transferred to the forward stay.

The foregoing results indicate a hinge action at the panel points due to the three-faced joints, despite the resistance of the splice material.

The closing of the joints, at top and bottom, took place mainly between Stages 3-*H* and 2-*H*, during which stages the splices were riveted, which indicates a gradual re-adjustment of stress at the joint as the drift-pins were replaced one by one by rivets.

Similar results were obtained at the other panel points observed.

In every case there was a movement of nearly $\frac{1}{8}$ in. from the initial measurement, indicating practically a complete closing of the joint.

This closing of the joints implies a greater compression, by $\frac{1}{8}$ in., in the middle fibers than in the outer fibers of the member. As a result, the average stress over the middle third of the section tends to be materially higher than the average stress in the extreme top and bottom fibers. The splice material across the joints resists this effect until the strains in the splice are released by the replacing of the drift-pins by rivets.

These conclusions are substantiated by the final measurements, which yielded the following results:

	Stage 3–*II*.	Stage 2–*II*.
Average top and bottom fiber stress......	6 050	10 400
Average middle fiber stress..............	6 050	13 100

The final difference between extreme and middle fiber stress thus amounts to 2 700 lb. per sq. in. at the sections of measurement. The gauge points were approximately at the quarter points of the length between panel points; a larger difference, of course, would be found if the sections could be taken nearer to the ends of the members.

Summary of Conclusions.

1.—The Howard strain gauge is well adapted for the measurement of stresses in steel structures under quiescent load, provided that the stresses are less than 1 000 lb. per sq. in. With careful manipulation it is possible to determine the true stresses within less than 200 lb. per sq. in.

2.—A comparison between calculated and measured primary stresses reveals, in addition to the observational errors, an average difference of 5 per cent. This is due principally to variations in the loading and cross-sections from the values assumed, and indicates the futility of excessive refinement in the ordinary computation of stresses.

3.—The extreme variation of fiber stress from the average stress in a member was found to range from about 1 600 lb. per sq. in. with the lowest primary stresses to about 2 500 with the highest primary stresses. A part of this variation represented the secondary stress from vertical bending; the greater part, however, was due to the effects of lateral bending, shop inaccuracies, temperature strains, splice details,

non-uniform material, and other causes which are omitted from consideration in the computation of secondary stresses.

4.—Combining the maximum discrepancy between calculated and measured stress (5% + 1 000) with the maximum variation of extreme fiber stress (12% + 2 600), and allowing for the fact that in the Hell Gate Arch the three-faced joints tended to reduce the secondary stresses, it appears that about 25% + 4 000 lb. per sq. in. is a necessary margin to be deducted from the minimum elastic limit of the material in order to obtain the limiting safe working stress for bridge members under average conditions.

5.—During erection, the secondary stresses—restricting the meaning of the term to the effect of bending in the plane of the truss—had an average value of 1 050 lb. per sq. in., calculated, and only 700 lb. per sq. in., measured. The highest calculated secondary stress was 2 920, and the corresponding measured secondary stress was 1 990 lb. per sq. in. Except for the smallest secondary stresses, the measured values were consistently lower than the calculated values. For the highest percentages of secondary stresses, the measured values are only a small fraction of the calculated values.

It is believed that similar results, though not as marked, would be found in other structures. The actual secondary stresses will generally be lower than the calculated values. There is an automatic re-adjustment of strains within a structure in such direction as to relieve the secondary stresses.

6.—The variations of calculated and measured secondary stresses in the successive erection stages are plotted for comparison in Fig. 5. The graphs show the measured stresses lower than the calculated values throughout the erection. The differences between the two curves are explained by special conditions in the erection of the structure.

7.—In the early erection stages, the three-faced joints (Fig. 6) between the lower chord members acted as hinges to permit a certain amount of rotation, so as to ease the secondary stresses. In the succeeding stages, as the direct stress increased, the joints became compressed, and the rotation was restricted. Between Stages 6 and 8, when there was a decrease in the direct compression, rotation occurred again, and the secondary stresses were partly released.

Extensometer measurements across the splice openings confirmed the above-described hinge action of the joints.

Another object of the three-faced joints was to produce a concentration of pressure in the middle third of the section, accompanied by a reduction of direct stress in the outer fibers. This distribution of stress, with the largest intensities in the middle third, was confirmed by actual measumement in the final stage.

8.—Measurements across the splice openings (Fig. 6) before and after riveting showed a complete closing of the joints during this operation. This is proof of the release of strains at a joint when the drift-pins are replaced by rivets.

The closing of the openings represents a desired transfer of initial stress from the splice material to the butt joint; it is also visible evidence of the greater concentration of stress in the middle third of the cross-section.

9.—The curve of calculated secondary stresses shows a large drop from the cantilever to the three-hinged stage, on account of the large reduction in deflections. This indicates the comparative freedom of the arch type from secondary stresses.

10.—In the final stage, the stresses in the extreme fibers, representing the maximum combined effect of primary and secondary stresses, show a remarkable agreement with the calculated values of the same stresses.

Acknowledgments.

Acknowledgments, in addition to those already given, are due to Mr. Frank E. Berry and to Theodore Belzner, Assoc. Am. Soc. C. E., for instruction and suggestions in the use of the extensometer; to Messrs. Brown and Sharpe, manufacturers of the instrument, for courtesies extended; to O. H. S. Koch, Jun. Am. Soc. C. E., who, with assistants, took all the measurements, and on whose painstaking thoroughness their value depended; to Mr. F. de Schauensee, who calculated the secondary stresses and assisted in reducing the observations; and to O. H. Ammann, M. Am. Soc. C. E., for suggestions during the prosecution of the investigation.

APPENDIX

SECONDARY STRESSES IN ARCH TRUSSES OF HELL GATE BRIDGE.

In view of the unusual size and form of the trusses of the Hell Gate Arch Bridge and for comparison with stress measurements, a complete analysis was made of the secondary stresses from dead load, live load, and temperature changes, in the finished structure, as well as during the various stages of erection.

The writer selected Winkler's analytical method, for these computations, in preference to the graphic method of Professor Mohr, because the greater precision and ease of supervision appeared to be sufficient compensation for the slight increase in time required. Except for some changes in the arrangement and tabulation of the computations, the procedure given in Johnson, Bryan, and Turneaure's "Modern Framed Structures" was closely followed. The calculation of secondary stresses for each load condition involves the solution of a series of simultaneous equations equal in number to the total number of panel points in the truss, and the expediting of the entire computation depends largely on the method selected for solving these equations. The quickest and best results were obtained with a modification of the method of successive approximations described by F. E. Turneaure, M. Am. Soc. C. E.* The modification consisted in substituting, in the first and each successive solution, the new values of the unknowns as far as already obtained, instead of using the values from the preceding approximation.

The computation of the secondary stresses from dead load on the three-hinged arch were simplified by treating the two halves of the arch, up to the temporary crown hinge, as separate frames. The effect of friction at the hinges was neglected.

For the dead-load secondary stresses in the two-hinged arch, because of symmetry, only one-half of the truss had to be computed. The load for this case consisted of the concrete floor and tracks, which were added after the center top chord was connected.

In computing the secondary stress for live load, one-half of the arch was considered loaded, as this load produces nearly the maximum primary stress in most members. A simple reversal of the diagram and algebraic addition of the two sets of stresses gives the secondary stresses for live load covering the full span.

The results of the secondary stress computations for the Hell Gate Arch are recorded on Plates XXXVII, XXXVIII, and XXXIX. These plates also contain all data necessary for reproducing the computations.

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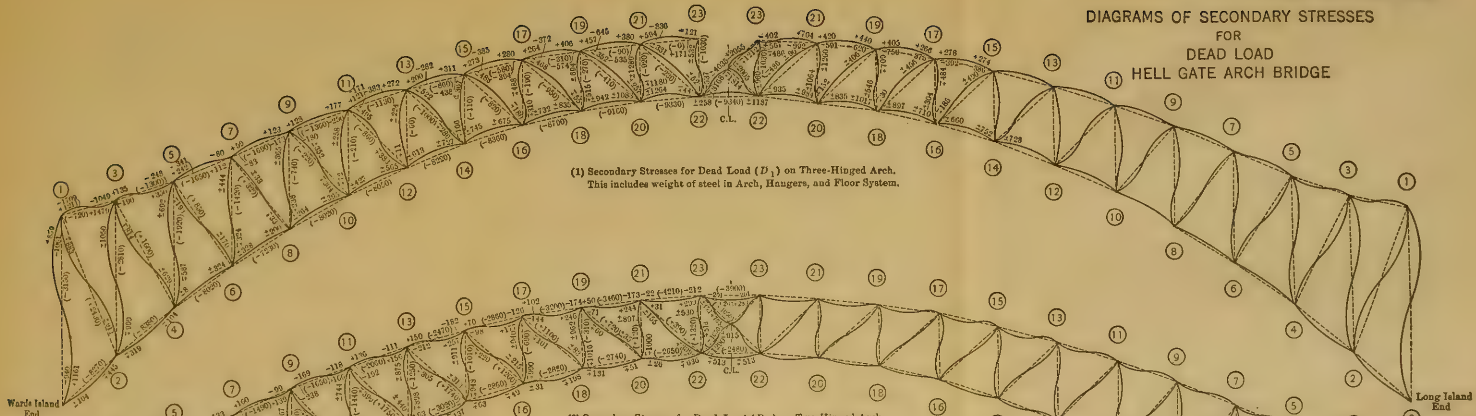
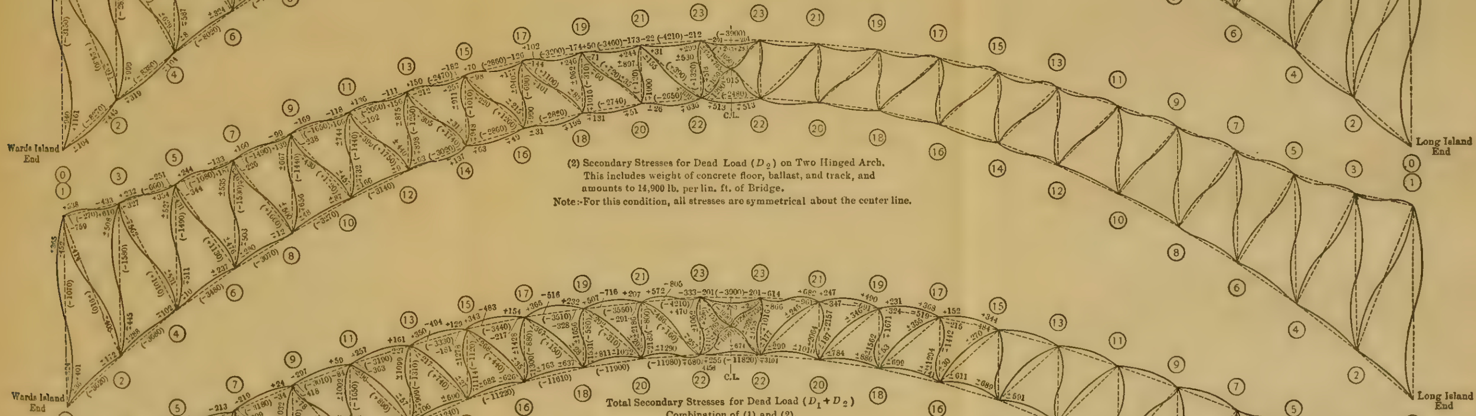
* *Engineering News*, September 5th, 1912.

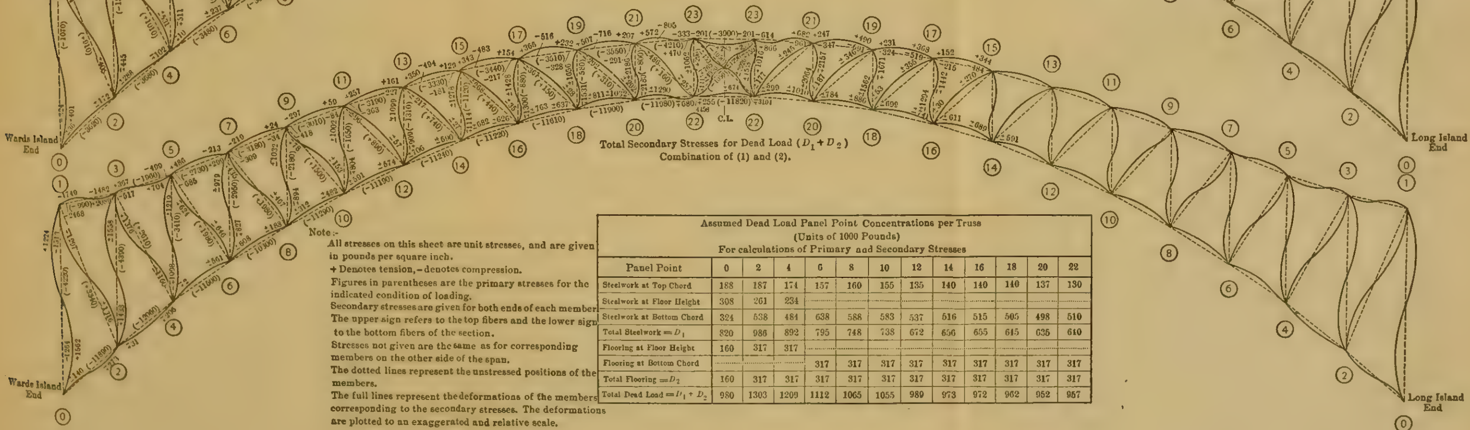
DIAGRAMS OF SECONDARY STRESSES FOR DEAD LOAD HELL GATE ARCH BRIDGE



(1) Secondary Stresses for Dead Load (D_1) on Three-Hinged Arch.
This includes weight of steel in Arch, Hangers, and Floor System.



(2) Secondary Stresses for Dead Load (D_2) on Two Hinged Arch.
This includes weight of concrete floor, ballast, and track, and amounts to 14,900 lb. per lin. ft. of Bridge.
Note:-For this condition, all stresses are symmetrical about the center line.



Total Secondary Stresses for Dead Load ($D_1 + D_2$)
Combination of (1) and (2).

Note:-
All stresses on this sheet are unit stresses, and are given in pounds per square inch.
+ Denotes tension, - denotes compression.
Figures in parentheses are the primary stresses for the indicated condition of loading.
Secondary stresses are given for both ends of each member.
The upper sign refers to the top fibers and the lower sign to the bottom fibers of the section.
Stresses not given are the same as for corresponding members on the other side of the span.
The dotted lines represent the unstressed positions of the members.
The full lines represent the deformations of the members corresponding to the secondary stresses. The deformations are plotted to an exaggerated and relative scale.

Assumed Dead Load Panel Point Concentrations per Truss
(Units of 1000 Pounds)
For calculations of Primary and Secondary Stresses

Panel Point	0	2	4	6	8	10	12	14	16	18	20	22
Steelwork at Top Chord	188	187	174	157	160	155	135	140	140	140	137	130
Steelwork at Floor Height	308	261	234	......	......	......	......	......	......	......	......	......
Steelwork at Bottom Chord	324	538	484	638	588	583	537	516	515	505	498	510
Total Steelwork $= D_1$	820	986	892	795	748	738	672	656	655	645	635	640
Flooring at Floor Height	160	317	317	......	......	......	......	......	......	......	......	......
Flooring at Bottom Chord	......	......	......	317	317	317	317	317	317	317	317	317
Total Flooring $= D_2$	160	317	317	317	317	317	317	317	317	317	317	317
Total Dead Load $= D_1 + D_2$	980	1303	1209	1112	1065	1055	989	973	972	962	952	957

The diagrams give, for each designated condition of loading, the secondary stresses at both ends of each member. A double sign prefixed to each figure indicates the kind of stress in the top and bottom fibers of the section, respectively. In addition, the primary stress for the same condition of loading is marked on each member in parentheses.

Another feature in these diagrams is the graphic representation, for the individual members, of the deformations corresponding to the secondary stresses. These deformations are shown exaggerated to an arbitrary relative but non-proportional scale. After the curves are drawn they serve principally as a general indication of relative distortions. In addition, the sharpness of curvature at any point represents the intensity of bending moment; the points of contraflexure mark points of zero secondary stress; the direction of curvature gives the signs of the secondary stress; the configuration of curves meeting at any panel point determines the direction of deflection of that point; and, in general, the deformation curves afford a visual check on the correctness of the work.

Results of the Secondary Stress Calculations.

An inspection of the secondary stresses recorded on Plate XXXVII yields the following facts:

The largest dead-load secondary stresses, in pounds per square inch, are as follows:

	Lower Chord.	Upper Chord.	Diagonals.	Verticals
	1 290 in 20-22	2 468 in 1-3	4 950 in 22-*M*	2 186 in 20-21
	1 032 in 18-20	961 in 21-23	1 376 in 3-4	1 671 in 18-19
			1 316 in 1-2	
All the other stresses are below	800	800	1 300	1 600

It will be observed that the largest secondary stresses in each class of members occur in the end panels and crown panels of the span. This effect is accounted for by the large concentrations of stress at Panel Points 0 and 22 of the three-hinged arch. It will generally be found that the largest secondary stresses in any structure occur where there is an interruption in the continuity of the truss configuration, or in the uniformity of the loading conditions.

Plate XXXVII also affords a comparison of the secondary stresses in a three-hinged and a two-hinged arch. The former are generally the larger.

In the outstanding dead-load secondary stresses it will be found that the major contributions to the total stress come from the three-hinged condition. This effect is most marked in the panels near the temporary crown hinge, as shown by Table 6.

TABLE 6.

Member.	Secondary stress from dead load on 3-hinged arch, in pounds per square inch.	Total secondary stress from dead load, in pounds per square inch.
18–20	1 083	1 032
20–22	1 264	1 290
21–23	992	961
22–*M*	4 035	4 950
M–23	2 055	1 290

The loading added in the two-hinged condition, although amounting to nearly one-half of the dead load on the three-hinged arch, does not produce its proportional share of the total secondary stress. In many cases it even helps to neutralize or reduce the secondary stress originating in the three-hinged condition.

An inspection of the secondary stresses on Plate XXXVIII yields the following facts:

As was to be expected from the respective deformations, the secondary stresses for live load covering the half span are considerably greater than for the live load covering the full span.

With only two or three exceptions in each case, the secondary stresses for full live load are less than 400 in the low chord, 500 in the upper chord, 800 in the diagonals, and 1 600 in the verticals. These low values indicate the eminent freedom of the arch from secondary stresses under full or uniform loading.

The second diagram on Plate XXXVIII is similar in all respects to the second diagram on Plate XXXVII, and the stresses are found to be proportional. This affords a check on the correctness of the computations, as the respective stresses were found by entirely different and independent procedures.

Plate XXXIX presents a compilation of all the secondary stresses and a comparison with the corresponding primary stresses. The data on this plate are self-explanatory.

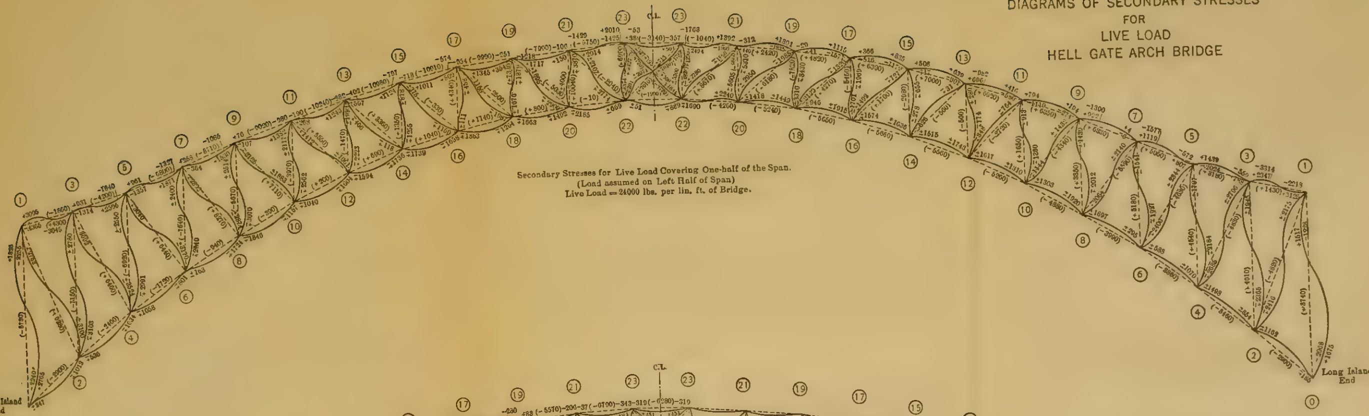
The secondary stresses of greatest critical interest are those occurring with the loading that produces the maximum primary stress in each member. The total secondary stresses for such condition, including both the dead and live-load contributions, are tabulated in Column 5. These secondary stresses, with a few exceptions in each group, are found to be less than 1 500 for the lower and upper chord members, less than 3 000 for the diagonals, and less than 3 500 for the verticals.

The last column gives the maximum values of the secondary stresses expressed as percentages of the corresponding primary stresses.

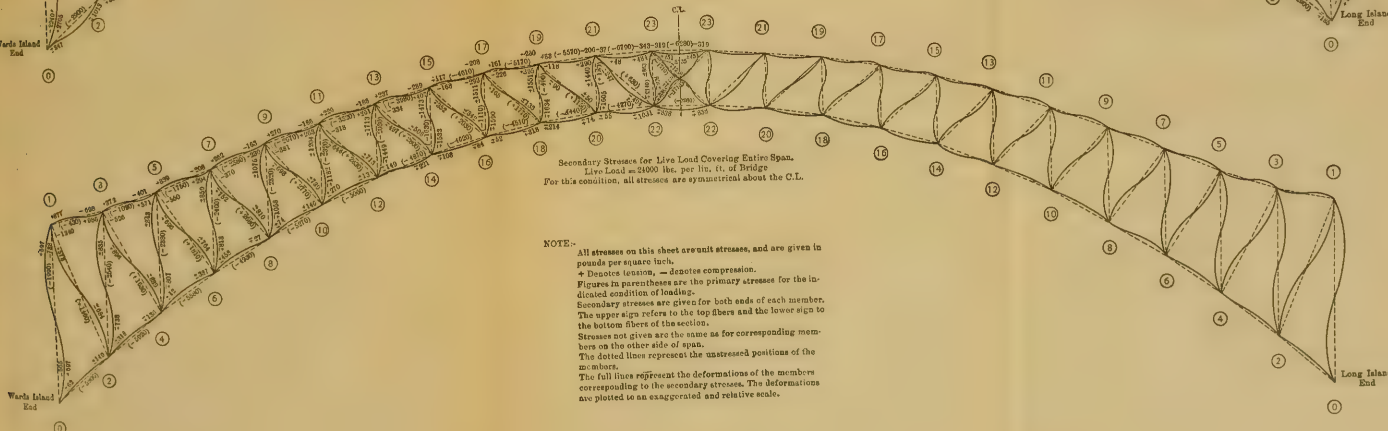
It will be observed that most of the percentages, especially those in the chord members, are small.

PLATE XXXVIII.
PAPERS, AM. SOC. C. E.
OCTOBER, 1917.
STEINMAN ON
SECONDARY STRESSES IN
HELL GATE ARCH BRIDGE.

DIAGRAMS OF SECONDARY STRESSES FOR LIVE LOAD HELL GATE ARCH BRIDGE



Secondary Stresses for Live Load Covering One-half of the Span.
(Load assumed on Left Half of Span)
Live Load = 24000 lbs. per lin. ft. of Bridge.



Secondary Stresses for Live Load Covering Entire Span.
Live Load = 24000 lbs. per lin. ft. of Bridge
For this condition, all stresses are symmetrical about the C.L.

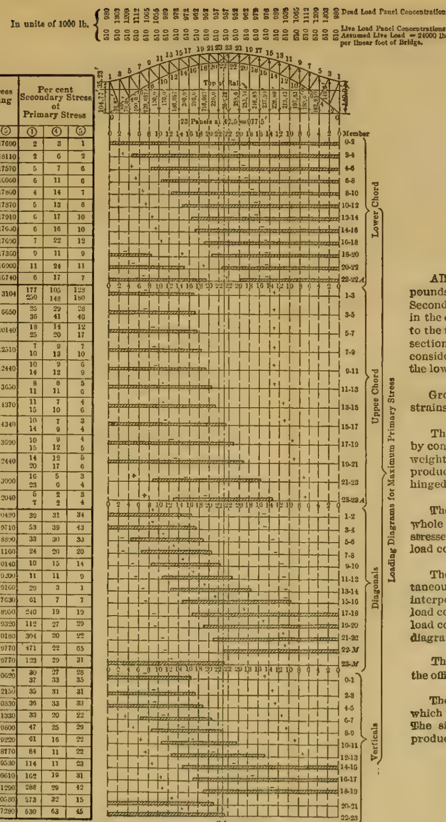
NOTE:-
All stresses on this sheet are unit stresses, and are given in pounds per square inch.
+ Denotes tension, — denotes compression.
Figures in parentheses are the primary stresses for the indicated condition of loading.
Secondary stresses are given for both ends of each member. The upper sign refers to the top fibers and the lower sign to the bottom fibers of the section.
Stresses not given are the same as for corresponding members on the other side of span.
The dotted lines represent the unstressed positions of the members.
The full lines represent the deformations of the members corresponding to the secondary stresses. The deformations are plotted to an exaggerated and relative scale.

PLATE XXXIX.
PAPERS, AM. SOC. C. E.
OCTOBER, 1917.
STEINMAN ON
SECONDARY STRESSES IN
HELL GATE ARCH BRIDGE.

TABLE OF SECONDARY STRESSES HELL GATE ARCH BRIDGE.

	Member	Constants: Length, In.	Constants: Cross Section, In.²	Constants: Moment of Inertia, In.⁴	Constants: Extreme Fiber Distance, In.	Secondary Stress From ① Dead Load: Wards L. Side		① Dead Load: Long L. Side		② Live Load Covering One-half Span: Loaded Side		② Live Load Covering One-half Span: Unloaded Side		③ Live Load Covering Whole Span		④ Approximate Secondary Stress Occurring Simultaneously with Max. Live Load Stress		⑤ Total ① + ④: Wards L. Side		⑤ Total ① + ④: Long L. Side		Primary Stress Corresponding To ①	Primary Stress ④	Primary Stress ⑤	Per cent Secondary Stress of Primary Stress ①	Per cent ④	Per cent ⑤
Lower Chord	0-2	674	1384	2310000	63.6	± 140	∓ 273	± 140	∓ 273	± 247	∓ 1013	∓ 185	± 1162	± 62	± 149	± 62	± 149	± 202	∓ 124	± 202	∓ 124	−11890	− 5800	−17690	2	3	1
	2-4	648	1281	1928000	61.4	∓ 31	∓ 206	∓ 31	∓ 206	∓ 536	∓ 1034	± 854	± 1498	± 318	∓ 136	± 382	± 60	± 351	∓ 146	± 351	∓ 146	−12060	− 6050	−18110	2	6	2
	4-6	624	1265	1857000	59.6	∓ 2	± 561	∓ 2	± 561	∓ 1058	∓ 201	± 1070	± 583	± 12	± 387	± 300	± 442	± 298	± 1003	± 298	± 1003	−11500	− 6070	−17570	5	7	6
	6-8	602	1337	1943000	58.4	± 608	± 188	± 608	± 188	± 163	∓ 1724	± 295	± 1697	± 458	∓ 27	± 396	± 628	± 1004	± 816	± 1004	± 816	−10300	− 5760	−16060	6	11	6
	8-10	582	1163	1461000	55.4	± 312	± 482	± 312	± 482	∓ 1846	∓ 1157	± 1920	± 1303	± 74	± 146	± 931	± 682	± 1243	± 1164	± 1243	± 1164	−11290	− 6570	−17860	4	14	7
	10-12	564	1121	1250000	52.6	± 591	± 574	± 591	± 574	∓ 1040	∓ 1604	± 1310	± 1617	± 270	± 13	± 828	± 874	± 1419	± 1448	± 1419	± 1448	−11190	− 6680	−17870	5	13	8
	12-14	548	1080	1050000	49.4	± 706	± 690	± 706	± 591	∓ 1594	∓ 1736	± 1743	± 1515	± 149	∓ 221	± 1104	± 818	± 1810	± 1408	± 1810	± 1409	−11240	− 6670	−17910	6	17	10
	14-16	535	1046	971000	47.6	± 682	± 626	± 689	± 611	∓ 1739	∓ 1658	± 1636	± 1574	∓ 103	∓ 84	± 1037	± 1004	± 1719	± 1630	± 1726	± 1615	−11220	− 6410	−17630	6	16	10
	16-18	524	988	867000	45.5	± 763	± 637	± 741	± 699	∓ 1863	∓ 1264	± 1915	± 946	± 52	∓ 318	± 1345	± 559	± 2108	± 1196	± 2086	± 1258	−11610	− 6080	−17690	7	22	12
	18-20	516	946	764000	43.4	± 811	± 1032	± 860	± 784	∓ 1663	∓ 1492	± 1440	± 1418	∓ 214	∓ 74	± 532	± 595	± 1343	± 1627	± 1418	± 1379	−11900	− 5460	−17360	9	11	9
	20-22	512	929	690000	41.9	± 1200	∓ 680	± 1010	± 299	∓ 2185	± 659	± 2240	∓ 1690	± 55	∓ 1031	± 580	∓ 1189	± 1870	∓ 1869	± 1590	∓ 890	−11080	− 4920	−16000	11	24	11
	22-22A	510	929	690000	41.9	∓ 255	± 674	± 674	∓ 255	± 51	∓ 889	∓ 889	± 51	± 838	∓ 838	∓ 846	∓ 696	∓ 1101	∓ 22	∓ 172	∓ 951	−11820	− 4920	−16740	6	17	7
Upper Chord	1-3	523	315	98000	19.5 27.5	+ 1740 − 2468	− 1482 + 2089	+ 1740 − 2468	− 1482 + 2089	+ 3095 − 4365	− 3045 + 4300	− 2218 + 3125	+ 2347 − 3314	+ 877 − 1240	− 698 + 980	+ 2215 − 3130	− 2180 + 3080	+ 3964 − 5598	− 3662 + 5169	+ 3964 − 5598	− 3662 + 5169	− 990	− 2114	− 3104	177 250	105 148	123 180
	3-5	535	315	98000	19.5 27.5	+ 367 − 517	− 499 + 704	+ 367 − 517	− 499 + 704	+ 931 − 1314	− 1840 + 2595	− 559 + 788	+ 1430 − 2024	+ 372 − 526	− 401 + 571	+ 695 − 980	− 1371 + 1931	+ 1062 − 1497	− 1870 + 2635	+ 1062 − 1497	− 1870 + 2635	− 1960	− 4690	− 6650	25 36	29 41	28 40
	5-7	550	315	98000	19.5 27.5	+ 480 − 685	− 213 + 299	+ 486 − 685	− 213 + 299	+ 961 − 1357	− 1327 + 1871	− 572 + 807	+ 1119 − 1577	+ 389 − 550	− 208 + 294	+ 746 − 1051	− 1030 + 1452	+ 1232 − 1736	− 1243 + 1751	+ 1232 − 1736	− 1243 + 1751	− 2730	− 7410	−10140	18 25	14 20	12 17
	7-9	553	330	110000	19.5 27.5	+ 219 − 309	+ 24 − 34	+ 219 − 309	+ 24 − 34	+ 258 − 364	− 1085 + 1530	+ 4 − 6	+ 922 − 1300	+ 262 − 370	− 163 + 230	+ 210 − 295	− 881 + 1241	+ 429 − 604	− 857 + 1207	+ 429 − 604	− 857 + 1207	− 3180	− 9330	−12510	7 10	9 13	7 10
	9-11	543	386	116000	19.5 27.5	+ 297 − 418	+ 60 − 84	+ 297 − 418	+ 59 − 84	+ 76 − 107	− 950 + 1332	+ 194 − 274	+ 794 − 1119	+ 270 − 381	− 186 + 263	+ 64 − 90	− 828 + 1169	+ 361 − 508	− 769 + 1085	+ 361 − 508	− 769 + 1085	− 3010	− 9430	−12440	10 14	9 12	6 9
	11-13	535	386	116000	19.5 27.5	+ 257 − 363	+ 161 − 227	+ 257 − 363	+ 161 − 227	− 190 + 268	− 882 + 1243	+ 416 − 586	+ 696 − 982	+ 226 − 318	− 186 + 261	− 170 + 240	− 788 + 1111	+ 87 − 123	− 627 + 884	+ 87 − 123	− 627 + 884	− 3190	−10460	−13650	8 11	8 11	5 6
	13-15	527	386	116000	19.5 27.5	+ 350 − 494	+ 129 − 181	+ 350 − 494	+ 129 − 181	− 402 + 567	− 797 + 1124	+ 639 − 901	+ 508 − 717	+ 237 − 334	− 289 + 407	− 378 + 534	− 750 + 1059	− 28 + 40	− 621 + 875	− 28 + 40	− 621 + 873	− 3330	−11040	−14370	11 15	7 10	4 6
	15-17	521	386	116000	19.5 27.5	+ 343 − 483	+ 154 − 217	+ 344 − 484	+ 152 − 215	− 718 + 1011	− 574 + 809	+ 835 − 1179	+ 366 − 516	+ 117 − 168	− 208 + 293	− 718 + 1011	− 574 + 809	− 375 + 528	− 420 + 592	− 374 + 527	− 422 + 594	− 3440	−10900	−14340	10 14	7 9	3 4
	17-19	516	386	116000	19.5 27.5	+ 366 − 516	+ 232 − 328	+ 368 − 519	+ 231 − 324	− 954 + 1345	− 251 + 354	+ 1115 − 1571	− 29 + 41	+ 161 − 226	− 280 + 395	− 885 + 1247	− 269 + 378	− 519 + 731	− 37 + 50	− 517 + 728	− 38 + 54	− 3510	−10180	−13690	10 15	9 12	4 6
	19-21	51.	386	116000	19.5 27.6	+ 507 − 716	+ 207 − 291	+ 490 − 691	+ 247 − 347	− 1218 + 1717	+ 106 − 150	+ 1301 − 1835	− 312 + 440	+ 81 − 118	− 206 + 290	− 1062 + 1497	+ 69 − 97	− 555 + 731	+ 276 − 388	− 572 + 806	− 316 + 441	− 3550	− 8890	−12440	14 20	12 17	5 6
	21-23	511	330	110000	19.5 27.5	+ 572 − 805	− 333 + 470	+ 682 − 961	− 614 + 866	− 1429 + 2014	+ 1425 − 2010	+ 1392 − 1966	− 1768 + 2494	− 37 + 48	− 343 + 484	− 404 + 570	+ 283 − 400	+ 168 − 235	− 50 + 70	+ 278 − 391	+ 331 − 446	− 4210	− 8830	−13090	16 23	5 0	3 4
	23-23A	510	330	110000	19.5 27.5	− 201 + 283	− 201 + 283	− 201 + 283	− 201 + 283	+ 38 − 53	− 357 + 504	− 357 + 504	+ 38 − 53	− 319 + 451	− 319 + 451	− 137 + 194	− 137 + 194	− 338 + 477	− 338 + 477	− 338 + 477	− 338 + 447	− 3900	− 8140	−12040	5 7	2 2	3 4
Diagonals	1-2	1340	216	80000	27	∓ 1207	± 1316	∓ 1207	± 1316	∓ 2753	± 3100	± 2175	∓ 2410	∓ 578	± 684	∓ 1970	± 2220	∓ 3267	± 3536	∓ 3267	± 3536	+ 3440	+ 7140	+10480	39	31	34
	3-4	1083	228	76000	27	∓ 1376	± 1160	∓ 1376	± 1160	∓ 3692	± 3524	± 2796	∓ 2685	∓ 896	± 839	∓ 2790	± 2665	∓ 4166	± 3825	∓ 4166	± 3825	+ 2610	+ 7100	+ 9710	53	39	43
	5-6	911	202	52000	24	∓ 624	± 646	∓ 624	± 646	∓ 3040	± 3170	± 2344	∓ 2406	∓ 696	± 764	∓ 1983	± 2070	∓ 2607	± 2716	∓ 2607	± 2716	+ 1930	+ 6910	+ 8890	33	30	30
	7-8	818	130	28500	21	∓ 435	± 467	∓ 435	± 467	∓ 2892	± 2874	± 2140	∓ 2064	∓ 752	± 810	∓ 1805	± 1792	∓ 2240	± 2259	∓ 2240	± 2259	+ 1980	+ 9180	+11160	24	20	20
	9-10	768	130	28500	21	∓ 78	± 153	∓ 78	± 153	∓ 2125	± 1883	± 1432	∓ 1144	∓ 603	± 739	∓ 1300	± 1150	∓ 1378	± 1303	∓ 1378	± 1303	+ 1500	+ 8640	+10140	10	15	14
	11-12	729	130	28500	21	± 96	± 57	± 96	± 57	∓ 1412	± 1061	± 764	∓ 348	∓ 648	± 713	∓ 831	± 663	∓ 785	± 720	∓ 785	± 720	+ 890	+ 8310	+ 9200	11	11	9
	13-14	705	130	28500	21	± 217	± 27	± 217	± 27	∓ 460	∓ 118	∓ 31	± 620	∓ 497	± 503	∓ 214	± 102	± 3	± 129	± 8	± 120	+ 740	+ 8420	+ 9160	29	3	1
	15-16	654	169	30700	21	± 208	± 35	± 270	± 30	± 375	∓ 1150	∓ 727	± 1492	∓ 352	± 342	∓ 318	± 530	∓ 60	± 565	∓ 48	± 560	+ 440	+ 7190	+ 7630	61	7	7
	17-18	675	169	30700	21	± 367	± 28	± 355	± 53	± 1155	∓ 1087	∓ 1320	± 2120	∓ 165	± 133	∓ 1039	± 1639	∓ 672	± 1537	∓ 684	± 1692	+ 150	+ 8800	+ 8950	240	19	19
	19-20	615	106	36000	21	± 292	± 292	± 346	± 187	± 1895	∓ 2900	∓ 1985	± 2950	∓ 90	± 50	∓ 1658	± 2450	∓ 1366	± 2742	∓ 1313	± 2637	+ 310	+ 9010	+ 9320	112	27	29
	21-22	684	106	36000	21	± 496	∓ 257	± 245	± 177	± 2107	∓ 2774	∓ 1860	± 2230	± 247	∓ 494	∓ 1683	± 2046	∓ 1197	± 1789	∓ 1438	± 2223	+ 160	+10020	+10180	304	20	22
	22-M	351	130	28400	21	∓ 4458	± 4950	∓ 3104	± 2920	∓ 294	± 2074	∓ 1684	∓ 598	∓ 2088	± 1476	∓ 1884	∓ 593	∓ 6342	± 4352	∓ 4938	± 2322	− 1050	− 8720	− 9770	471	22	65
	23-M	351	130	28400	21	± 453	∓ 765	∓ 760	± 1200	± 2527	∓ 1954	∓ 1792	± 716	± 735	∓ 1238	± 2527	∓ 1954	± 2989	∓ 2719	± 1768	∓ 664	− 1050	− 8720	− 9770	123	29	31
Verticals	0-1	1680	312*	110000*	27.5 34	− 1264 + 1562	+ 1224 − 1513	− 1264 + 1562	+ 1224 − 1513	− 2240 + 2765	+ 1625 − 2255	+ 1675 − 2068	− 1228 + 1517	− 565 + 697	+ 597 − 738	− 1709 + 2105	+ 1390 − 1720	− 2973 + 3667	+ 2614 − 3233	− 2973 + 3667	+ 2614 − 3233	− 4290*	− 6400*	−10620*	30 37	27 33	28 35
	2-3	1355	259*	94000*	27	∓ 1438	± 1558	∓ 1438	± 1558	∓ 3103	± 2780	± 2365	∓ 1945	∓ 739	± 835	∓ 2380	± 2128	∓ 3818	± 3686	∓ 3818	± 3686	− 4390*	− 7760*	−12150	35	31	31
	4-5	1116	235*	70000*	27	∓ 1098	± 1219	∓ 1098	± 1219	∓ 2091	± 2550	± 2184	∓ 1707	∓ 807	± 843	∓ 2442	± 2082	∓ 3540	± 3301	∓ 3540	± 3301	− 3410	− 7420	−10830	36	33	33
	6-7	960	151	36000	21	∓ 827	± 979	∓ 827	± 979	∓ 2740	± 2400	± 1927	∓ 1541	∓ 813	± 859	∓ 1710	± 1500	∓ 2537	± 2479	∓ 2537	± 2479	− 2950	− 8380	−11330	33	20	22
	8-9	854	137	31000	21	∓ 894	± 1032	∓ 894	± 1032	∓ 3070	± 2514	± 2012	∓ 1430	∓ 1058	± 1075	∓ 1920	± 1572	∓ 2814	± 2604	∓ 2814	± 2604	− 2180	− 7620	− 9800	47	25	29
	10-11	761	126	27000	21	∓ 804	± 1002	∓ 804	± 1002	∓ 2562	± 2117	± 1350	∓ 913	∓ 1182	± 1204	∓ 1181	± 1006	∓ 1985	± 2008	∓ 1985	± 2008	− 1650	− 7570	− 9220	61	16	22
	12-13	682	126	27000	21	∓ 909	± 1099	∓ 909	± 1099	∓ 2223	± 1993	± 774	∓ 280	∓ 1449	± 1713	∓ 738	± 809	∓ 1647	± 1908	∓ 1647	± 1908	− 1310	− 7460	− 8770	84	11	22
	14-15	615	126	27000	21	∓ 1114	± 1278	∓ 1114	± 1278	∓ 1235	± 818	∓ 276	± 655	∓ 1533	± 1473	∓ 654	± 900	∓ 1768	± 2178	∓ 1768	± 2178	− 1120	− 8410	− 9530	114	11	23
	16-17	552	126	27000	21	∓ 1300	± 1429	∓ 1294	± 1424	± 111	∓ 458	∓ 1701	± 1969	∓ 1590	± 1511	∓ 1673	± 1855	∓ 2973	± 3283	∓ 2967	± 3279	− 880	− 9730	−10610	162	19	31
	18-19	622	126	27000	21	∓ 1531	± 1656	∓ 1562	± 1671	± 1676	∓ 1879	∓ 3310	± 3430	∓ 1634	± 1551	∓ 3029	± 3113	∓ 4560	± 4769	∓ 4591	± 4784	− 580	−10710	−11290	288	29	42
	20-21	490	126	27000	21	∓ 2180	± 2180	∓ 2064	± 2157	± 4000	∓ 3270	∓ 5606	± 5310	∓ 1605	± 1440	± 3064	∓ 3540	± 1464	∓ 1354	± 1600	∓ 1383	− 800	+11380	+10580	273	32	15
	22-23	483	126	27000	21	∓ 1116	± 1062	∓ 1538	± 1016	± 4374	∓ 3427	∓ 5300	± 4290	∓ 926	± 863	± 4374	∓ 3427	± 3259	∓ 2365	± 2836	∓ 2411	+ 290	+ 6990	+ 7280	630	63	45

* Average Value



Note:-The above loading diagrams refer to members in the left half of span only. For corresponding members in the right half of span, the loading diagrams should be reversed.

NOTES

All stresses on this sheet are unit stresses, and are given in pounds per sq. in. + denotes tension, − denotes compression. Secondary stresses are given for both ends of each member in the order of the designating numbers. The upper sign refers to the top fibers and the lower sign to the bottom fibers of the section. In verticals the fibers toward the springing are considered as the upper fibers and those toward the crown as the lower fibres.

Gross sections were used for figuring unit stresses and strains.

The secondary stresses ① from dead load were obtained by combining the secondary stresses produced by the steel weight acting on the 3-hinged arch with the secondary stresses produced by the weight of floor and track acting on the 2-hinged arch.

The secondary stresses ③ from live load covering the whole span, were obtained by combining the secondary stresses ② in the loaded and unloaded sides produced by live load covering one-half of the span.

The approximate secondary stresses ④ occurring simultaneously with maximum live-load stress were obtained by interpolation between the secondary stresses ② from live load covering one-half of the span and those ③ from live load covering the whole span, in accordance with the loading diagrams appended to the table.

The primary stresses given on this sheet were figured from the official Table of Stresses for the Hell Gate Arch.

The loading diagrams indicate by shaded areas the loading which produces maximum primary stress in each member. The signs (+) or (−) indicate the character of the stress produced by any loads in the shaded or unshaded distances.

In the lower chord, the critical secondary stresses range from 1 to 12% of the maximum primary stresses.

In the upper chord, the first three members have comparatively high percentages of secondary stresses (180, 40, and 17%), but this is due to the low primary stresses (3 104, 6 650, and 10 140, respectively) in these members. There is an excess of section which amply provides for these secondary stresses. The totals of secondary plus primary stress in these three members do not reach 12 000 lb. per sq. in.

In the other upper chord members the critical secondary stresses range from 3 to 10% of the primary stresses.

In the web members the critical secondary stresses range from 1 to 65%, but only a few exceed 30 per cent. There is ample margin of cross-section to take care of these stresses.

The web members, as a rule, have larger secondary stresses than the chord members. This may appear surprising, as the web members are longer and narrower than the chord members, and it is generally stated that the secondary stresses in different members vary inversely as their slenderness ratios. The reason is due to the form of the arch; in the three-hinged condition practically all the dead load is carried by the parabolic bottom chord, and the web members receive large secondary stresses from the compression of the lower chord members before they receive any appreciable primary stress.

Conclusions.

These results obtained in the secondary-stress computations bear out the justification of the type of structure, unit stresses, and method of erection adopted for the Hell Gate Arch Bridge.

Some of the larger secondary stresses are found at the crown of the arch, and appear to be caused by the special arrangement of the center panel and the part which it plays in the erection of the structure as a three-hinged arch. There were sufficient advantages in this construction, however, to outweigh the increase in the local secondary stresses. Nevertheless, the results confirm the usual objections to intersecting web members. On the other hand, they demonstrate the desirability, from the consideration at least of secondary stresses, of reducing the number of hinges in an arch.

The low secondary stresses in the bottom chords are particularly gratifying, as they set at rest any apprehensions aroused by the great depth of these members.

The results, on the whole, show that special methods of assembling and erection, such as were adopted for the Quebec and Sciotoville Bridges, to counteract the effect of secondary stresses, were not required for the Hell Gate Arch Bridge.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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PAPERS AND DISCUSSIONS

THE RECONSTRUCTION OF THE STONY RIVER DAM

Discussion.*

By F. W. Scheidenhelm, M. Am. Soc. C. E.†

F. W. Scheidenhelm,‡ M. Am. Soc. C. E. (by letter).§—The discussion of the paper has been very interesting to the writer, as presumably it has also been to other engineers concerned with the problems treated. Among other things, it serves to emphasize the fact that there is rarely agreement among engineers as to the exact measure, or margin, of safety which should be given to, or is possessed by, a structure. On the whole, the writer finds much cause for satisfaction in the comments contained in the discussions. Mr. Scheidenhelm.

Frequently, papers presented before the Society and the discussions thereon result in a symposium of data and opinion on certain subjects. Such is the case with regard, in particular, to two problems which were involved in the reconstruction of the Stony River Dam, and are treated in the paper.

The first of these problems concerns the determination of the maximum flood for which spillway capacity is to be provided, and the consequent determination and design of the spillway provision itself. Of the ten members who have discussed this matter, only one, Mr. Gregory, feels that the writer has "surely gone beyond the necessary limit." The great majority, however, apparently agree on the proposition that a drainage area as small as that of Stony River at the dam site requires entirely different treatment than do larger

* Discussion of the paper by F. W. Scheidenhelm, M. Am. Soc. C. E., continued from August, 1917, *Proceedings*.

† Author's closure.

‡ New York City.

§ Received by the Secretary, September 28th, 1917.

Mr. Scheidenhelm. areas, and that the exceptionally large maximum run-off adopted, although surprising, is warranted.

Referring to the comment of Mr. Gregory (page 923*) the writer feels that it is certainly necessary to take into account, not merely rainfalls of 2 or 3 hours' duration, but those of considerably longer periods, and more especially their consequent floods. This is for the reason that from such rainfalls the run-off (into the reservoir) which immediately precedes the maximum or critical periods of, say, 2 or 3 hours, necessarily affects the absorption or equalization capacity of the reservoir during the vital period.

It is true, as pointed out in the paper, that a record or graph of run-off never truly represents the corresponding rainfall; that is, the maximum rate of run-off is always less than the maximum intensity of rainfall. Yet, even making full allowance for this fact, as applied to actual Stony River conditions, it is reasonably evident, from the studies depicted in Figs. 12 and 13, that the length of the period of rainfall which should be considered is at least 6 hours; and it must be remembered that rainfall does not persist at a uniform intensity through any 24-hour period. Thus, were there an automatic gauge record available for the 22-in. rainfall during 24 hours at Altapass, N. C., on June 15th-16th, 1916 (page 184†), the record for the maximum 6 hours would probably be astounding.

Again, looking at the matter from Mr. Gregory's point of view, and making the reasonable assumption, size and topography of drainage area considered, that "the peak of the run-off at the dam is equal to the greatest average rate of rainfall during any 45-min. period" (page 194†), it is to be noted from Fig. 15 that "the most severe run-off conditions reasonably conceivable", there shown, indicate a rainfall with an intensity averaging only about 4 in. per hour for the maximum 45 min. This intensity is somewhat less than that given for the same period by the Talbot formula, quoted by Mr. Gregory. and, of course, is less, by an even greater amount, than the corresponding "Maximum for Greater New York" (Fig. 32). Incidentally, the Talbot and the Greater New York curves of Fig. 32, when plotted on Fig. 9, fall outside of the group of curves shown thereon; in other words, the curves of Fig. 32 show the greater intensities of rainfall.

It is interesting to note that apparently the Cane Creek and Elkhorn Creek floods, which formed the basis for the studies presented in the paper (page 180†) were of such intensity as not to be included within the formula proposed by Weston E. Fuller, M. Am. Soc. C. E.* The same is true also of a run-off of 1 302 cu. ft. per

* *Proceedings,* Am. Soc. C. E., May, 1917.

† *Proceedings,* Am. Soc. C. E., February, 1917.

‡ "Flood Flows," *Transactions,* Am. Soc. C. E., Vol. LXXVI, p. 564.

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sq. mile, reported* for the drainage area of 143 sq. miles of Devil's Creek, near Viele, Iowa.

The method of determining or checking spillway capacity by means of graphs of hypothetical maximum floods to be provided against, seems to meet with approval. Similar methods are shown to have been used in the design work of the Miami Conservancy District at Dayton, Ohio, as referred to by Mr. Riegel (page 1301†), and discussed in considerable detail by Mr. Grant (page 930‡). A remarkably close agreement of results is evident from comparison of Figs. 12 and 36, the former being based on the method used by the writer, whereas the latter is based on the flood routing method applied on the work of the Miami Conservancy District.

The second feature provoking considerable discussion is the utilization of the underlying foundation soil in setting up resistance to sliding. On this subject the discussion of Professor Cain is a contribution of exceptional value.

One may fairly question whether present experimental information regarding the behavior of soils is sufficient to warrant the acceptance of Professor Cain's application of Coulomb's laws to cases, such as that of the Stony River Dam, where there is involved the question of sliding, and more particularly of impending sliding. In fact, the writer still feels constrained to distinguish between the conditions existing before a body of clay has failed in shear along a given plane or surface and the conditions existing after the resistance to shear has been overcome (that is, when, in his terminology, cohesion has given way to friction, plus any adhesion). Such distinction does not appear to be inconsistent with Coulomb's laws, especially as expressed by Professor Cain somewhat more broadly in his paper on "Cohesion in Earth."§ In fact, Professor Cain's more frequent use of the term "shear" in his analyses based on Coulomb's laws, as applied in his discussion (page 908‡), is significant. Even Professor Cain now considers that possibly not all "cohesion" (Cain) is destroyed on the beginning of the actual motion or sliding.||

However that may be, all must be in full agreement with Professor Cain in his emphasis of the "need for comprehensive experimentation to determine the coefficients of cohesion." Moreover, the writer believes that Professor Cain has performed an important service in presenting in such detail the several analyses of the Stony River

* *Water Supply Paper No. 162*, U. S. Geological Survey.

† *Proceedings*, Am. Soc. C. E., August, 1917.

‡ *Proceedings*, Am. Soc. C. E., May, 1917.

§ *Transactions*, Am. Soc. C. E., Vol. LXXX, p. 1316.

|| This appreciable part of "cohesion" exerted in addition to friction during the actual sliding of clay on clay (p. 908), the writer would term "adhesion", as more accurately describing its nature.

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problem on the basis of coherent earth. It is not improbable that, as the result of such work as is now being carried on by the Society's Special Committee on Soils, the treatment advanced by Professor Cain may be adopted, at least in part, and especially as applied to cases of actual sliding.

The writer agrees entirely with Professor Cain that probably the effect of a longer time of application of a given shearing load on a body of clay under test would be to afford lower values of unit shearing resistance (page 909*). It should be noted, however, that, in the Stony River design, no reliance was placed on the shearing resistance of the foundation soil.

In the interesting analyses (pages 910-911*) of the original structure before failure, based on the theory of coherent earth, Professor Cain unwittingly made several erroneous assumptions.

The mistake apparently was caused by his treatment of the section of dam shown on Plate V as being taken at the maximum height portion of the dam. This is not the case, for Plate V shows the "Typical Section at Bay 35" referred to in Columns (3), (5), (7), and (9) of Table 3. The resulting error in the value of Q, as derived by Professor Cain (page 910*), is not serious, but, unfortunately, the same mistake made later in his discussion robs the results of their significance, though, of course, it does not deprive the theoretical analysis of its value as such.

Thus, in assuming anchoring walls applied to the original structure, as shown on Plate V, Bay 35, Professor Cain perforce used loads (page 912*) as given in the writer's Cases I and II (pages 210-212†) and applying to the sections of maximum height, instead of to the section at Bay 35. The difference in height of sections will be apparent on reference to Plate II. Moreover, the loads used by Professor Cain apply to the structure in its original condition, whereas both concrete and water weights were greatly increased in the reconstruction, without any increase in the horizontal water thrust (Table 3). The horizontal water thrust for Case I, for instance, is but 53 820 lb. per lin. ft., instead of 81 000 lb., as assumed by Professor Cain.

In consequence, the conclusions as to resistance to sliding, furnished under the respective hypotheses set up under the theory of coherent earth, are not applicable. The corrected figures show much larger margins of safety. Nevertheless, it appears to the writer that the appropriateness of the methods used by Professor Cain has in no wise been affected; and, for the purpose of theoretical study, those interested may well disregard the inapplicability of the figures.

Professor Cain neglects the resistance along BC and IJ of his Fig. 28, on the ground that there is "but little weight over parts of

* *Proceedings*, Am. Soc. C. E., May, 1917.

† *Proceedings*, Am. Soc. C. E., February, 1917.

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these surfaces" (page 917*). It should be evident, however, on study of Plate V and the description of the toe-wall and its effect (page 218†), that the new toe-wall in essence extends the base of the dam as far as F (Fig. 28) and that, in fact, the greatest vertical loads on the soil occur at the very down-stream edge of the toe-wall, namely, just to the left of the plane represented by the line, FC. For this reason the writer believes that one is not warranted in omitting from consideration the resistance along BC and IJ.

The graphical solution for the passive resistance of the down-stream foundation soil is exceedingly interesting, and here again those interested incur debt to Professor Cain. The point made by him, that the writer was too conservative in assuming zero load at the free surface of the earth at F (Fig. 28), is well taken, as is evident from the analyses (pages 920-921*).

On the subject of the safety of the structure, as repaired and strengthened, the measures taken appear to have general approval, except by Mr. Rutenberg. In view of the nature of the discussion submitted by him (pages 653-659‡), the writer is loath to make a detailed reply. However, a number of the premises adopted by him diverge so widely from the facts that it appears advisable to examine some of the points which he has raised, in order properly to co-ordinate the paper and the discussion thereof. Except in special instances, the writer will refrain from discussing mere expressions of opinion appearing in Mr. Rutenberg's contribution, for, of course, each engineer is entitled to his own opinion, and it would be futile to attempt to obtain agreement on all points:

1.—With reference to the discussion on remedying the faulty condition of the foundation soil (page 654*), the writer desires to emphasize that reliance against leakage was placed solely on the extension and perfecting of the cut-off wall, and not on the compacting of the foundation soil by grouting. The object of the grouting, as stated, it is believed clearly, on pages 241-242†, was to compact the soil for bearing purposes. In other words, it was to remedy conditions caused by leakage which occurred prior to the failure of the dam; or, in the words of the desideratum expressed by Mr. Rutenberg himself, "to fill, if possible, all the holes found in the foundation soil" (page 657‡). It will be noted that the subject of pressure grouting was treated by the writer primarily under the heading of "Footings", and not under that of "Leakage and Drainage."

In passing, it may not be out of place to remark that it is the writer's experience that pressure grouting in porous soils and gravels

* *Proceedings*, Am. Soc. C. E., May, 1917.

† *Proceedings*, Am. Soc. C. E., February, 1917.

‡ *Proceedings*, Am. Soc. C. E., April, 1917.

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is not a sure means of causing water-tightness, but that, on the other hand, such grouting is generally effective in rock having pronounced seams.

2.—The writer agrees in essence with Mr. Rutenberg's statements in the third paragraph of page 654,* but cannot agree with his conclusion as expressed in the paragraph following. The writer must still insist that "the smaller the coefficient of frictional resistance, the less the net effect of uplift pressure." Perhaps this anomaly of agreement on certain premises and disagreement as to the conclusion, is explicable on the ground that Mr. Rutenberg has not noted that the writer's statement applies solely to the effect of uplift pressure in so far as resistance to sliding is concerned. This is manifest from the context (page 201†).

The meaning of the writer's statement can perhaps be made more clear by using a numerical example. Thus, if over a given area of base of dam there should exist an uplift pressure of 100 000 lb., there would result the equivalent of a net reduction in weight by the same amount. The net reduction in weight, however, is a different thing from reduction in resistance to sliding. For instance, if in this hypothetical case the coefficient of frictional resistance were 0.75, the resistance to sliding would, by reason of the assumed uplift, be decreased by 75 000 lb. In the case of the Stony River foundation soil, however, the coefficient of frictional resistance probably averages about 0.33. This coefficient, applied to the uplift condition just assumed, would result in decreasing the resistance to sliding by only 33 000 lb.

3.—The expression of opinion that leakage and percolation in the dam should have been intercepted "directly and immediately on the up-stream face of its cut-off wall, and discharged down stream by special channels without any pressure" (page 654*) is so unique and, in the writer's opinion, illogical, that he hesitates to believe that it was intended to make such a statement. The writer insists that one need be concerned only with such water (leakage) as passes through the diaphragm (intended to be water-tight) of which the cut-off wall is a part. Of course, the primary effort was devoted to making the cut-off wall as tight as possible. In this regard, the results of the reconstruction speak for themselves, namely, that at present the merest trickles of water pass through or under those portions of the dam founded on earth over-burden.

4.—Although desiring to avoid useless quibbling over terms, it seems necessary to point out that the statement imputed to the writer, "that water under pressure in the foundation soil is never harmless" (page 655*), is not warranted by the statement that, in the case of the Stony River foundation, deep-lying pockets, if any, of water under

* *Proceedings*, Am. Soc. C. E., April, 1917.

† *Proceedings*, Am. Soc. C. E., February, 1917.

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pressure are "relatively harmless" (page 251*). By "deep-lying pockets of water" were meant accumulations of water, under pressure, existing at elevations below the bottom of the new anchoring wall at the heel; in other words, below the probable location of the main "planes of least resistance." Such deep-lying pockets of water are relatively harmless, partly because of their depth, and partly because of the comparatively small coefficient of frictional resistance obtaining in the Stony River overburden, the effect and relation of which coefficient to uplift pressure and resistance to sliding have been explained in Paragraph 2.

5.—The evidently unintentional misstatement is made that the writer "admits the presence of considerable hydrostatic pressures at a distance of 20 ft. from the down-stream side of the cut-off wall" (page 655†). The writer's actual statement, as given on page 252*, is believed to be self-explanatory to a careful reader. The writer does not admit the presence of such considerable hydrostatic pressures; on the contrary, he desires again to point out that no pressure head of more than 2 ft. has been detected at the top of any weep-holes or drain pipes since the reconstruction was completed (pages 253-254*).

6.—The premises being erroneous, as shown above, Mr. Rutenberg arrives at a conclusion manifestly erroneous, namely, that "the up-stream half of the base is exposed to dangerous uplift pressures" (page 655†). This erroneous conclusion leads to a likewise erroneously applied train of "previously stated consequences" (for which see page 655†).

7.—Again, there is imputed to the writer cognizance of the actual existence of the rather strained hypothetical phenomena which Mr. Rutenberg sets up as to the possibility that "the drainage pipes will be filled through the higher perforations from the higher strata of the foundation soil, etc." (page 655†). It is conceded that such conditions might exist, but, if so, it is far from certain that the results would be harmful. Be that as it may, however, the phenomenon referred to by the writer (page 254*) clearly lends no weight whatsoever to the hypothetical conditions proposed.

Relative to this matter, it should be noted that the fact that so many of the weep-holes and drain pipes remain just full of water, but not over-flowing, is sufficient proof that the source of such water is ground-water, and that it is not the result of leakage from the reservoir. The writer is inclining more and more to a belief in the correctness of the explanation of the phenomenon on the basis of capillarity.

* *Proceedings*, Am. Soc. C. E., February, 1917.

† *Proceedings*, Am. Soc. C. E., April, 1917.

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8.—From a casual inspection of Plate V, Mr. Rutenberg ventures to pass judgment as to the strength of the anchoring wall at the heel. That the inspection must have been casual is shown by the fact that, contrary to his statement (page 656*), Plate V does give the essential dimensions, and even the details, of the reinforcement of the anchoring wall at Bay 35. Furthermore, data as to the anchoring walls at Bays 30 and 31 are shown on Plate VI.

An exposition of the detailed calculations involved in the design of the anchoring walls might have been included in the paper. However, even as it stands, the paper has been characterized as exhaustive. The addition of such an exposition, therefore, did not seem advisable, and at this date the military duties of the writer preclude such diversion on his part. Hence it must suffice to state that the anchoring walls at heel and toe were designed, with standard factors of safety, so as to carry, jointly and safely, all the horizontal thrust, excepting only an amount equal to the product of the weight of the superstructure (plus water load) multiplied, in the case of the clayey soil, by a coefficient of frictional resistance of 0.2 (page 232†). The latter amount of horizontal thrust can certainly be counted on to be transferred into the foundation soil directly by the frictional resistance existing between the soil and the base of the superstructure. Essentially the same method has been used by Professor Cain, in the admirable analysis contained in his discussion (page 914‡). The only difference is that Professor Cain there considers the writer's "frictional resistance" as subdivided into friction and "cohesion."

As a matter of fact, the results obtained by Professor Cain for the load carried directly from the base of the dam into the earth, and thence down to the "planes of least resistance", without being transferred through the anchoring walls at the heel or toe, are very much greater than those obtained by the writer by the use of the coefficient, 0.2. It will be noted that the coefficient assumed by the writer for this particular purpose is only 80% of the "probable minimum value" shown in Table 1; and, as such, it is believed to be thoroughly conservative, with the purpose and result that the anchoring walls were correspondingly built so much the stronger.

9.—The fear is expressed that the toe-wall "will aid in collecting the leakage under the dam" (page 656*). Inasmuch as weep-holes were left through the footings (see Plate V), it is not believed that the hydrostatic pressure in the foundation soil at the toe-wall can be sufficient (the depth of the wall considered) to require relief through the toe-wall. An exception exists in the case of the old spillway.

* *Proceedings*, Am. Soc. C. E., April, 1917.

† *Proceedings*, Am. Soc. C. E., February, 1917.

‡ *Proceedings*, Am. Soc. C. E., May, 1917.

where the toe-wall is especially deep (17 ft.). There, drainage openings were provided through the vertical member of the toe-wall, the details of which are shown in Fig. 27. Mr. Scheidenhelm.

10.—With reference to the effect of the heel-wall and its "under-cut slope" (page 657*), the attention of Mr. Rutenberg is respectfully called to the physical fact that the heel-wall cannot possibly present additional surface for the application of uplift pressure without at the same time presenting additional surface for the application of downward water pressure, which, of course, resists overturning. Even granting that the up-stream portion of the heel-wall is subjected to full hydrostatic pressure on the "under-cut slope", the net effect of the heel-wall would still be to resist overturning, for the reason that the specific gravity of the heel-wall concrete is much greater than that of the water which it displaces. In this respect the location of the heel-wall so far up stream, and its correspondingly great lever arm, are of no small consequence in actually increasing the stability of the structure against overturning. Manifestly, the result is diametrically opposite to that stated in the discussion.

11.—The writer quite agrees with the implication regarding the tests of frictional resistance of various Stony River soils, to the extent that it would be dangerous to rely on these as giving truly representative results (page 657*). However, it by no means follows that the tests are "without practical value." On what is our knowledge of the frictional resistance of soils based, if not on observation and tests, as well as on theory? It should have been unnecessary again to point out what was evidently overlooked, namely, that the writer discounted the test results to such a degree that, instead of assuming an average coefficient of 0.61, he based the design on a coefficient of 0.33 (page 207†). Moreover, the structure is shown actually to be safe with a coefficient of 0.25.

12.—Similarly, it should not have been necessary to remind any one that no pretense has been made by the writer that the weight of the underlying foundation soil aids, in the least, to resist overturning. The weight of the underlying foundation soil is utilized solely to resist failure by sliding. Of course, once such soil has been washed away, it can no longer resist sliding; but failure resulting from such conditions would have been due primarily to leakage or undermining; and against failure of that sort entirely different precautions have been taken.

13.—Mr. Rutenberg is under a misapprehension as to the writer's "adding 3½ ft. of water to the original level in the reservoir" (page 657*). As a matter of fact, the writer did not add to the water level, but merely made provision for taking care of floods, which would have

* *Proceedings*, Am. Soc. C. E., April, 1917.

† *Proceedings*, Am. Soc. C. E., February, 1917.

Mr. Scheidenhelm. come with equal probability in the cases of the original and the strengthened structure. True, a parapet was added to prevent overflow at the bulkhead sections and consequent undermining at the down-stream edges of those sections. However, that the additional horizontal water pressure caused thereby is negligible should be patent to every one with a moment's computation. On the other hand, the new spillway serves to hold down the water level in the reservoir for a given flood discharge, as compared with the water level which would have existed in the case of the original structure for the same flood discharge. It is pertinent to note that, for instance, even on the basis of a run-off of 1 368 sec-ft. per sq. mile (as reported for Cane Creek, page 180*), the bulkhead section of the original structure would have been overtopped by $2\frac{1}{2}$ ft. That is, such a discharge could have taken place only after the water level had risen to an elevation of about 141.5. The corresponding water level under the new conditions for a spillway discharge of 1 386 sec-ft. per sq. mile will be approximately at Elevation 141.65. Evidently, the difference is negligible.

Again, reference to Table 3 will show that the same elevation of head-water, namely 140.5, was used for the stability analyses for both the original and the strengthened structures under "normal maximum load."

14.—It is true that the superstructure of the new spillway section is heavier than that of the original portions of the dam. However, it does not follow, by any means, that either "the original structure is insufficient or the excess of the new is useless" (page 658†). As to the original structure, the writer was forced to choose, on the one hand, between recommending the expenditure of a relatively enormous sum of money in order to strengthen the decks and buttresses, and, on the other hand, accepting somewhat higher working stresses than would have been allowed had the structure been designed initially for the water loading for which it should have been designed. As to the new spillway, the increased cost of using normal working stresses, as compared with the cost for the slightly higher stresses obtaining in the original portions of the dam, for the same loading, was not great, and there would have been no justification for holding down the new spillway dimensions to correspond with those of the original structure (page 259*). A situation of this sort requires the use of common sense as well as the application of theory.

15.—The unit bearing stresses for the soil and the base of the dam, the necessary data for the computation of which Mr. Rutenberg misses (page 658†), are set forth in considerable detail on pages

* *Proceedings*, Am. Soc. C. E., February, 1917.

† *Proceedings*, Am. Soc. C. E., April, 1917.

232-236*. Further details might have been added had it been desirable to extend indefinitely the limits of the paper. Mr. Scheidenhelm.

16.—The writer is unable to agree that the reservoir or groundwater leakage, mentioned on page 254* and discussed on page 658†, has found other opportunities for "harmful erosion." Mere differences of opinion are immaterial, but it is to the point to know that since about a month after the reservoir had completely filled, there have been no further appearances of muddy water at any of the many drainage openings in the dam. This fact would seem to bear out the writer's opinion (page 254*) that the muddy nature of the water which appeared was due to adjustments in the foundation soil to the new conditions of loading.

The foregoing comments have been essentially on divergences from the facts. However, certain mere expressions of opinion on part of Mr. Rutenberg seem to warrant passing mention. For instance, he "believes the reconstructed dam is less safe than the original structure" (page 658†) and "is firmly convinced that the reconstructed dam will also be broken, perhaps soon, if adequate repairs are not made in time" (page 659†). It is unfortunate that opinions of so grave a nature and predictions so dire should be advanced, based, as they are, on erroneous premises, and certainly not substantiated by the facts. Probably the most important considerations in this regard are the results of the reconstruction, which speak for themselves:

a.—In the first place, the reconstructed dam has now stood for more than 2 years under full water load. The original structure stood only 65 days under full water load prior to failure.

b.—Shortly after the reconstruction was completed, the dam was subjected to a flood which caused the head-water to rise about 3½ ft. higher than the highest water level which had occurred during the short life of the original structure.

c.—Finally, an inspection made recently by the writer showed that the dam is in excellent condition.

Certain criticisms have been offered by Mr. Moore (page 659†). Two of these in effect duplicate criticisms made by Mr. Rutenberg, namely, as to the value of the writer's tests of frictional resistance and as to the structural strength of the new anchoring walls at the heel of the dam. The writer's comments in regard to these criticisms may be found in the paragraphs numbered 11 and 8, respectively.

The contingency of the fracture of the anchoring wall at the heel, to which Mr. Moore refers, is not believed by the writer to be a serious one. The extent to which such considerations are pertinent is a matter of judgment, as to which it is not surprising to find differences of opinion. Had it been considered necessary by the writer, the heel-

* *Proceedings*, Am. Soc. C. E., February, 1917.

† *Proceedings*, Am. Soc. C. E., April, 1917.

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wall and toe-wall could, and would, have been made even deeper, thus further reducing the unit soil pressures caused by such horizontal thrust as is transferred by the anchoring walls into the foundation soil. The writer desires, further, to point out that it would be impossible to have uplift pressures from both clay and water acting on the same areas of the base of the dam. Inasmuch, therefore, as certain allowances were made for the uplift pressure of water (Table 3), it is evident that considerable provision has already been made for flotation effect, if any, on part of the foundation soil.

The "floating off" of a dam is, in the writer's opinion, a fanciful expression rather than a serious possibility. He has yet to learn of an actual case of floating of a dam on either water or soil. Nevertheless, he does not mean to belittle the possibilities and effect of uplift pressure, as is shown by the load assumptions set forth in Table 3 under the "most severe conditions within limits of reason."

Again, Mr. Moore states that the water assumed to flow over the intermediate spillway would not fall "well-nigh" vertically. Although desirous of avoiding any hair-splitting discussion as to what "well-nigh" vertically means, the writer wishes to state that the upper and lower nappes of the over-falling water were determined according to the results of the experimentation of M. Bazin, and the tumbling hearth mentioned (page 185*) was designed accordingly. It is interesting to note that Mr. Brodie in his discussion (page 667†) mentions that his own checking gave results practically identical with those of the writer for the shape of the new spillway crest. The same method was used by the writer in determining the shape of the new spillway crest as in determining the nappes of the over-falling sheet of water at the intermediate spillway. The fact that both studies were based on the work of M. Bazin explains the close check between the results obtained by Mr. Brodie and by the writer.

The real value of the use of the brittle steel pins as flash-board supports is no longer a matter of doubt; the results of three seasons of operation speak for themselves. Thus far, the pins and flash-boards have allowed the undisturbed and safe utilization of the 25% additional capacity of the reservoir made available by the use of the flash-boards. The writer does not pretend that any and all brittle steel pins will accomplish such results, but does believe that merit is justly attributable to the pins adopted in the present case.

In addition to the foregoing, certain scattered comments appear to be appropriate to round out the closure of the discussion:

Mr. Finch very properly emphasizes the advisability of paying more attention to the architectural treatment of dams, etc. (page 652†).

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* *Proceedings*, Am. Soc. C. E., February, 1917.

† *Proceedings*, Am. Soc. C. E., April, 1917.

In this the writer entirely agrees, even though special treatment was not necessary or warranted in an isolated location like that of the dam under discussion. The type of treatment, of course, had been fixed before the writer's arrival on the scene. It is noteworthy, however, that the construction of a second spillway gave a balancing effect, especially to the down-stream view of the dam, which the original structure entirely lacked. Mr. Scheidenhelm.

With reference to Mr. Downs' discussion of the coefficients of frictional resistance of Appalachian materials (page 660*), it is well to remember that though the tests showed an average coefficient of 0.61, the designs were actually based on a probable coefficient of 0.33, and that it is designed to be safe for a coefficient of 0.25, without reliance on the shearing resistance of the clayey soil.

On the several questions raised by Mr. Dunham (page 663*), the writer is unable to shed any light. Most of the questions do not fall within the scope of his investigation, and it is not believed to be appropriate to extend the present discussion by a consideration of the possibilities of imitating Nature's regulation of the flow of streams. The use of the increased flow of the river for power purposes was not contemplated.

Mr. Gregory expresses concern as to the advisability of placing tar-paper in the joints of a dam (page 925†). In the case of this dam, this material was not expected to be permanent, and it is believed that, as the tar-paper gradually disappears, in years to come, it will simultaneously be replaced by sediment and accretions resulting from the action of the reservoir water on the concrete, particularly the alkaline elements thereof.

It was economy that dictated the use of such an expedient as tar-paper; yet its immediate effectiveness appears to have been as great as one could expect from any other method. The leakage through the joints has actually become negligible, and that, of course, is the result sought. A leakage proportionate to that reported by Mr. Gregory as having existed at the Ashokan Dam (page 929†) would have been intolerable at Stony River, in view of the relatively small reservoir capacity available in the latter case. It is interesting to note that the initial daily leakage at the Ashokan Dam was approximately one-quarter of the total storage capacity of the Stony River Reservoir. Possibly the reduction in leakage in the former case has likewise been due to sedimentation; such an effect has frequently been observed by the writer.

The percentage of "plums" specified in connection with the Class *C* concrete was intended to fix a maximum limit, and does not represent an average of the quantity actually used. As a matter of fact, a

* *Proceedings*, Am. Soc. C. E., April, 1917.

† *Proceedings*, Am. Soc. C. E., May, 1917.

Mr. Scheidenhelm.

maximum limit need hardly be fixed. At any rate, the writer agrees with Mr. Jorgensen that 20% of "plums" represents a very high average (page 934*). The writer does not know the actual quantity of "plums" placed in the Class *C* concrete at Stony River.

The mathematical solution presented (page 940*) by Professor Church, for determining portions of curves showing "discharge over spillways", such as used in Figs. 12 to 15, is indeed most interesting, utilizing, as it does, methods of attack in which Professor Church is a past master.

Unfortunately, quantitive measurements of leakage were not made on May 25th and November 1st, 1915, as referred to by Mr. O'Shaughnessy.

Mr. Justin offers an alternative suggestion for combining means to increase resistance to sliding and to increase the length of the path which leakage water from the reservoir must follow. The essentials of these means are shown in Fig. 39. With reference to the first phase of the suggestion, it may be said that preliminary studies along the same line, namely, of increasing the weight of superstructure (page 213†), showed that nothing short of a complete filling of the available space with masonry or concrete would suffice. Of course, the considerations regarding overloading of the foundation soil at the toe were applied to the base as actually, and materially, widened by the building of the new toe-wall. Had the expedient of increasing the superstructure weight been adopted, it would naturally have been advisable, as Mr. Justin suggests, to place the added weight as far up stream as possible. Unfortunately, however, the slope of the deck would have precluded obtaining any material advantage thereby.

A similar method of transferring the load from the buttress into the toe-wall was actually used in the reconstruction in connection with the special work at the outlet gate bays (see Plate VI).

As to the lengthening of the path of travel of leakage water, the writer believes that reliance may be placed on this method more safely in the case of gravel or sand foundations than where loam and clay are involved, as at the Stony River site. It should also be noted that the corresponding widening of the base of the dam, assuming an entire absence of drainage openings through the base, would increase the allowance necessary to be made for uplift pressure.

In the case of the suggested up-stream concrete apron, absolute water-tightness would be imperative, and therein, the writer believes, would lie the greatest difficulty, especially if the apron were made only 6 in. thick. Furthermore, the original cut-off wall is much less pervious than the soil which would underlie the suggested apron. Consequently, the result would probably be a gradual accumulation

* *Proceedings*, Am. Soc. C. E., May, 1917.

† *Proceedings*, Am. Soc. C. E., February, 1917.

of water under pressure under the apron, with resulting decrease in the efficacy of the expedient. This would be true even if the reservoir water could penetrate under the apron only at its up-stream edge. Mr. Scheidenhelm.

Although it was considered well worth while to make the expensive addition of anchoring walls to the original structure, it should be remembered that the same results could have been obtained much more easily and economically had a combination of anchoring wall at the heel and of cut-off been constructed as a part of the dam at the very beginning. For a case of original construction, therefore, the advantages of using the principle of anchoring walls are even more apparent.

The West Virginia Pulp and Paper Company is following the praiseworthy course of having annual inspections of the dam made by an experienced engineer; moreover, there is the additional safeguard of daily inspections by a regular attendant.

In conclusion, the writer wishes to express his satisfaction with the magnanimous approval contained in the discussions by Mr. Coburn, Chief Engineer of the Ambursen Company, and by Mr. Wegmann, who had been retained in an advisory capacity during the period when the original construction of the dam was under consideration.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

MODERN PRACTICE IN WOOD STAVE PIPE DESIGN AND SUGGESTIONS FOR STANDARD SPECIFICATIONS

Discussion.*

By Messrs. E. A. Moritz and A. N. Miller.†

E. A. Moritz,‡ Assoc. M. Am. Soc. C. E. (by letter).§—The author states that the primary purposes of his paper are: (*a*) to give engineers an idea of the difference between the various grades of wood pipes; (*b*) to set forth a standard set of specifications for the assistance of engineers who have had no opportunity to become versed in their design; and (*c*) to safeguard those who contemplate building such pipe. His purposes appear to have been very well accomplished and, on the whole, the paper is well worth careful study by any one who is without information or experience in wood stave pipe design. The writer would issue only one caution to the uninitiated, and that is, that the author appears to give undue prominence to the value of redwood as compared with fir, which may be due to the fact that he may have had less experience with the latter. This tendency will be discussed later. Mr. Moritz.

The writer will confine himself to pointing out some statements which are not exact or which are open to differences of opinion. One of these is that redwood is the best known material for wood pipe. This may have been true 25 years ago, but its accuracy, at the present time, is questioned. There are at least five large companies on the

* Discussion of the paper by J. F. Partridge, Jun. Am. Soc. C. E., continued from September, 1917, *Proceedings*.

† These discussions were presented before the Colorado Association of Members, Am. Soc. C. E., at its meeting of June 9th, 1917.

‡ Denver, Colo.

§ Received by the Secretary, September 25th, 1917.

Mr. Moritz.

Pacific Coast which manufacture fir pipe exclusively, and only two which manufacture redwood pipe. The writer has no statistics at hand, but it is not unlikely that the output of fir pipe far exceeds that of redwood.

In regard to redwood pipe, the author says:

"Direct exposure to the rays of the desert sun, and alternate wetting and drying when the pipe is used intermittently in irrigation systems, do not lessen its efficiency."

This appears to be an inaccurate statement, but if the author means that such conditions do not hasten the decay of the wood, he will find few engineers to agree with him.

The author cites various specifications of the U. S. Reclamation Service as examples of the embodiment of different fundamentals for the purchase of similar materials, and states that, in some of these, redwood is placed on an equal basis with uncoated fir and, in others, with coated fir. This is an incorrect understanding of the practice of the Reclamation Service, which is to compare redwood, coated fir, and uncoated fir on their merits, as influenced by the peculiar local conditions surrounding each installation. That is, when competitive bids are asked for the three different classes of materials, the prices submitted are not compared directly, but an estimate is made of the probable length of life of each class of construction and the cost of maintenance and replacement, and the ultimate cost is made the basis of decision as to the most economical construction. It is well known that redwood is much more durable than uncoated fir, and it is probable that it is more durable than coated fir under similar conditions, but, as to the latter, available information is not conclusive.

The statement that galvanized bands have been used on wood pipe in the Hawaiian Islands is interesting. The writer did not know that this had ever been done, and he has always held the opinion that the spelter could not withstand the severe treatment given the bands during erection. If galvanized rods can be put in place without serious injury to the spelter, such material would be a great improvement over the asphalt paint generally used, which is not satisfactory. The cost would not be prohibitive.

The author states that machine-banded pipe is made in sizes from 2 to 24 in. Several manufacturers are now making very satisfactory machine-banded pipe up to 30 in. in diameter. The writer is not informed as to whether or not such pipes have been manufactured in larger sizes than 30 in.

The author is not in favor of inserted or slip-joint pipe. Probably few, if any, engineers will agree with him in this. The inserted joint is entirely satisfactory for heads up to about 100 ft. and, if properly reinforced with individual bands, might be used successfully under higher heads.

The limit, no doubt, is lower for the larger than for the smaller pipe. Information on this point is not conclusive, but if the writer may venture a guess, it is that the reinforced inserted joint can be used successfully for 24-in. pipe under a maximum head of 50 ft. and for 8-in. pipe under a maximum head of 150 ft. The reinforced inserted joint has been used successfully on 30-in. pipe under a 25-ft. head. The wood collar is the most vulnerable part of the pipe to the ravages of decaying influences, due to its lack of saturation, and, on this account, it should be used only when necessary. Mr. Moritz.

The author says, "Fir is the pipe that has failed, the oldest lines having been built not more than 10 years, * * *". This is a surprising statement, in view of his knowledge of Mr. Henny's article on "Life of Wood Pipe."* This article does not, by any means, include all the wire-wound fir wood pipes that have been built, but, it does include one pipe that was 20 years old and another that was 19 years old in 1915. The former is stated to be in "good condition" and the latter "in excellent condition, including wire."

A. N. Miller,† Assoc. M. Am. Soc. C. E. (by letter).‡—This paper is a valuable addition to the existing literature on the subject of wood stave pipe. Mr. Miller.

The writer cannot agree with Mr. Partridge, however, when he states (page 583§) that machine-banded, Class *A* pipe "will have a life of from 15 to 25 years when receiving no attention; a longer life under ideal conditions, as when laid in soils having the least possible corrosive effect on the galvanized wire, and when operating under pressure, so as to insure complete saturation of the wood." This statement may or may not be true. It may be assumed that the life of clear redwood, completely saturated, is exceedingly long, and if the pipe is placed in a soil which permits of the staves being continually saturated, there is no doubt that they will last for 25 years, or even much longer.

The lack of permanency in machine-banded pipe is due to the corrosion of the wire, which is of comparatively small sectional area. As shown in Table 3, machine-banded pipe is at present manufactured in diameters up to 24 in., with sizes of wire varying from No. 12 in the 2-in. size to No. 4 in the 24-in. size, the spacing of the wires varying with the pressure, as there tabulated.

By referring to Table 2, on continuous wood stave pipe with ordinary band steel, it will be noted that the usual type is made successfully in diameters up to 144 in., and that the corresponding diameter of the band on the 24-in. pipe is $\frac{7}{16}$ in. The wire on the machine-banded

* *Reclamation Record*, August, 1915.

† Denver, Colo.

‡ Received by the Secretary, September 25th, 1917.

§ *Proceedings*, Am. Soc. C. E., April, 1917.

Mr. Miller. pipe is galvanized. On the ordinary pipe, the bars may or may not be; usually, they are not.

It has been the writer's experience that in the lighter gauges of metal, the corrosion of steel, when exposed to atmospheric and soil conditions, is extremely rapid, even though the metal is galvanized.

Much has been heard lately of the so-called pure irons, which are said to resist corrosion to a marked degree. The advertisements by the manufacturers of these materials would almost lead one to believe that the oxidation of iron and steel is a thing of the past. On close observation, one can readily see that these companies shield themselves behind a good coat of galvanizing. It has been stated recently by a representative of one of the largest manufacturers of this so-called pure iron, which, in reality, is a low-carbon steel made in an open-hearth furnace, that 60% of their tonnage is galvanized. The writer does not wish to give the impression that the product put out by these companies is inferior. He does wish, however, to emphasize the point that, in a practical sense, the virtue, or resistance of this metal to corrosion is due to the galvanized coating—not to the purity of the base metal, as represented in most of these advertisements. The skill of the press agents employed by these manufacturers is in advance of that of their chemists and artisans. It may be said that in the defeat of corrosion, purity in the base metal is undoubtedly desirable, but the requisite purity is practically or commercially unattainable.

When one studies the subject of corrosion from the point of view of the electrolytic theory, it is readily seen that it is very difficult to manufacture, on a commercial scale, a steel which is non-corrodible. It is practically impossible to eliminate all impurities; it is also difficult to render the steel perfectly homogeneous. As a result of the dissimilarity in the composition of the molecules, electric currents are set up, under certain conditions, which decompose any water present into its constituents, hydrogen and oxygen.

It may be considered that waters carried by wood stave pipes are weak electrolytes, due to the salts held in solution. It must be remembered, also, that ordinary waters are saturated with dissolved oxygen and carry carbonic acid in solution. These waters also have extremely variable degrees of alkalinity. Electrolysis and corrosion, therefore, go hand in hand, ever tending to restore these products to the more stable compounds found in Nature.

The writer has seen extra heavy wrought-iron pipe, in reality steel pipe, which was used to carry Pintsch gas to the railroad cars, rust out completely in 18 months. This pipe was laid in ashes in the railroad yards, and the conditions for rapid corrosion were ideal, the pipe being buried in a relatively concentrated electrolyte. This pipe failed, or rusted through, near the joints where the mechanics had applied their Stillson wrenches in screwing up the pipe, thereby break-

Mr. Miller.

ing the outer skin. This condition was finally corrected by embedding the pipes in cement mortar. Similar pipes in clay soil have been in use for 15 years without showing serious effects from oxidation.

In the purification of coal and water gas, hydrated oxide, or iron rust, is used to remove the sulphureted hydrogen. The oxide is made from cast-iron filings or machine-shop borings. The filings are piled and wet down with water, and a small quantity of common salt is usually added. In a very few hours great heat is evolved as the chemical action or oxidation proceeds. It is necessary to control the temperature of the mass, as the efficiency of the final product is lowered if high temperatures are permitted. This is done by spreading the oxide in thin layers. The mass is sprinkled from day to day with an ordinary garden hose. It is also turned over periodically so as to expose new surfaces to atmospheric influences. In the course of about 30 days or 6 weeks the oxide is ready for use. This oxide is found to be of slightly different chemical composition from the ordinary precipitated oxide. It is much cheaper, however. The writer cites the foregoing in order that those not familiar with the gas industry may know how quickly the complete oxidation of iron may take place.

It may be stated, therefore, that it is not well to fix arbitrarily the life of a wood stave pipe, at, say, from 15 to 25 years, without knowledge of its location and the chemical characteristics of the soils to which it may be subjected, as the life may be long or short, depending on these governing conditions.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

This Society is not responsible for any statement made or opinion expressed in its publications.

THE THREE 15-CUBIC YARD DIPPER-DREDGES, GAMBOA, PARAISO, AND CASCADAS, AS SUPPLIED AND USED ON THE PANAMA CANAL

Discussion.*

By Charles Evan Fowler, M. Am. Soc. C. E.

Charles Evan Fowler,† M. Am. Soc. C. E.—There has been no other kind of engineering design in which the practical problems have had larger scope than in that of dredges of all kinds, and particularly is this true of the design of dipper-dredges. For many years the speaker, as chief and consulting engineer, has been connected with the dredging of a vast quantity of material in the Pacific Northwest. This material has consisted of a greater percentage than usual of hardpan and cemented gravel, which is harder to dig than many kinds of soft rock, and offers greater resistance, for all parts of dredging machinery, than any material of which the speaker is aware. Mr. Fowler.

Some years ago, in building a dredge for digging this material, the machinery complete was purchased for a 7-yd. dipper plant, and, as the time in which to assemble the dredge was short, it was mounted on an old clam-shell dredge hull, 50 by 100 ft., and the compound engines on this were used as swinging engines by connecting them to the turn-table with double wire rope blocks. Many other makeshifts had to be used, but, after the rush work was completed, a new wooden hull was built, which was 40 by 100 by 11 ft., and proved to be none too large for dredging of this class. There is nothing more important than having a hull with sufficient beam for side digging, to take the load off the spuds, and, of course, the hull must be of such length

* This discussion (of the paper by Ray W. Berdeau, Jun. Am. Soc. C. E., published in August, 1917, *Proceedings*, and presented at the meeting of September 19th, 1917), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† New York City.

Mr. Fowler. that it will not rise out of the water at the stern when digging hard material or when digging with a large dipper. It will readily be seen from this that the first two Panama dredges, which had a beam of only 44 ft., were too narrow, and it was certainly a wise change to make the *Cascadas* of 55 ft. beam.

Even when ordinary conditions as to getting steel prevail, the speaker has found many advantages in building wooden hulls, inasmuch as it is often necessary to cut through the hull for various purposes, or to make various connections to it, all of which can be more readily done with a wooden hull. Then, again, a wooden hull is more easily repaired than a steel hull, and the speaker knows of wooden hulls which are 30 years old, are still in use, and are giving good service. However, such hulls must be properly built, so as to have sufficient strength and stiffness and plenty of air space; and, in salt water, they must be properly constructed for protection against the teredo. Under present conditions, the speaker thinks that, for all classes of dredges, hulls constructed of reinforced concrete would be cheaper than either wood or steel, and in many cases preferable. Many recent dredges of all types have been constructed with steel hulls, and some have been used on work with which the speaker has been connected, these dredges also having steel spuds and everything of the most substantial construction.

No matter how exactly everything has been figured out for dipper-dredges, especially as regards the boom and dipper handles, it will be found that it is necessary, almost immediately, to begin re-driving rivets and reinforcing the sections; therefore, it would seem to be only a matter of precaution to assume that the loads which cannot be figured should be taken care of by at least doubling the calculable stresses for loads suddenly applied. The dippers are the most frequent sources of trouble, and have to be reinforced, particularly on the front and around the nose. The same thing happens with almost all the clam-shell buckets of standard design, as it is usually necessary, not only to increase the weight of the shells, but of all the pins and bearings as well. With reference to sheaves, it has been the speaker's practice to make them as large as reasonably possible, in order to lengthen the life of the wire ropes, and to provide them with very large pins and boxes in order to avoid cutting the bearings.

The discussion on this paper has brought out the fact that there is grave doubt as to whether compound engines are desirable for dipper machines, and there is certainly no point about dredge design about which the purchaser should be more certain as to capacity than in the boilers and engines. The speaker has found it very satisfactory to check up the engines by Seaton's formula, which gives very close to the actual indicated horse-power, although it very often calls

for an engine of much larger size for the required horse-power than would be offered by an engine builder. Mr. Fowler.

For nearly twenty years the speaker has operated fleets of dredges, comprising all the different types, and it is largely a matter of judgment as to the type which should be used in any particular work. The plant of the United Dredging Company, for which the speaker is at present acting as Consulting Engineer, comprises suction hopper sea-going dredges; regular suction-dredges of the largest type; suction-dredges which are provided with propellers for going from one job to another under their own steam; dipper-dredges; and large clam-shell dredges. This gives the necessary variety to handle work of all kinds in the United States, Alaska, Mexico, Hawaii, and abroad.

The discussion has also brought out the fact that the cost given in the paper does not cover the cost of the disposal of the material, and this should be very strongly emphasized in the paper before its final publication, as it is very important to have dump-scows of proper number and size, as well as tugs of sufficient power, in order to get a project started, and take out the full yardage of which the dredge is capable. The speaker is always skeptical in regard to published costs of dredging work, as the figures are usually only for the bare dredging, without the proper overhead charges and extensive plant repair, which may cost as much as from $20 000 to $50 000 at a time, if a job has lasted from 6 to 12 months, and which certainly bears no relation to the ordinary figures which are included for simple depreciation.

Some years ago, when making prices on dredging work in the Northwest, the speaker was bothered with reports of dredging being done on the Columbia River at from 2 to 3 cents per cu. yd. This was investigated fully, the total costs for a number of years previous —back to the inception of the work—being taken from the reports of the Port of Portland, including overhead, experimental work, and expenditures of every sort. The yardage reported was also taken, although a large part of this was admittedly washed out by the current; and the cost was found to be more than 26 cents per cu. yd. This, of course, has been greatly bettered during recent years by more efficient administration and a more efficient and modern plant, but, from this source no more is heard of ridiculously low costs. Some one must pay general or overhead expenses eventually.

In closing, the speaker would like to call attention to a type of strongly built suction-dredge, provided with a good type of rock cutter, which can do very efficient work in material often excavated with a dipper-dredge or some other type. Modern high-powered suction-dredges with booster pumps in the pipe line have been used to pump to distances of 20 000 ft., or nearly 4 miles, and the modern design of dredging pumps, with from 1 200 to 1 500 h. p., are capable

Mr. Fowler. of pumping through very long pipe lines, ranging from 4 000 to 7 000 ft., without boosters; and to pump to very much greater heights than was believed possible a few years ago. In one recent case, the speaker delivered material with a pump of this type to a distance of about 4 500 ft. on a lift of 45 ft., the material being heavy sand with some clay. The places where dipper-dredges of very large size, or dredges of any type of large size, can be used economically are comparatively few, and the engineer must make sure of all his conditions before deciding on the type, or he must leave it to a contractor of great experience to select the kind of plant and let him take his own chances.

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PAPERS AND DISCUSSIONS

AIR TANKS ON PIPE LINES

Discussion.*

By Irving P. Church, Assoc. Am. Soc. C. E.

Irving P. Church,† Assoc. Am. Soc. C. E. (by letter).‡—All the algebraic relations and formulas of this interesting and well-arranged paper have been verified by the writer. In the derivation of Formula (R), however, it would seem that a special assumption is necessary, which is not mentioned in the paper. Without this assumption, the writer's analysis leads to an expression differing from the author's in a numerical coefficient, simply, involving a discrepancy of about 10 per cent. Mr. Church.

As of some possible interest, the writer herewith presents his analysis in the case of Formula (R).

As the calculus is to be used, it will be necessary to supplement and modify the author's notation somewhat, as follows:

Let t denote the time at any instant from the beginning of the (upward) motion of the water surface in the air tank;

Let T (and not t) denote the time for the pressure in the air tank to change from p_1 to p_2;

Let p be the air pressure (in feet of water), in the tank at any time; (and other notation as required).

The case of load thrown off, only, will be treated; air compressed. From Assumption 1, we have:

$$\frac{d\,p}{d\,t} = C' \text{ (a constant)}$$

* This discussion (of the paper by Minton M. Warren, Assoc. M. Am. Soc. C. E., published in August, 1917, *Proceedings*, but not presented at any meeting of the Society), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† Ithaca, N. Y.

‡ Received by the Secretary, September 10th, 1917.

Mr. Church.

and, therefore,

$$p = p_1 + C' t \dots\dots\dots\dots\dots\dots\dots\dots (1)$$

As Assumption 2 is not made, we have the relation,

$$A \frac{d y}{d t} = a (v - V_2) \dots\dots\dots\dots\dots\dots\dots\dots (2)$$

and, from Assumption 3 (that is, y is zero compared with p),

$$W a \cdot \frac{L}{g} \cdot \frac{d v}{d t} = - (p - p_1) W a \dots\dots\dots\dots (3)$$

Substituting in Equation (3) the value of p from Equation (1), there is obtained,

$$\frac{L}{g} \cdot d v = - C' t d t \dots\dots\dots\dots\dots\dots\dots\dots (4)$$

in which the variables v and t are separated.

By integrating Equation (4) between the limits v and V_1 for v, and t and zero for t, we find, after transposition and division by $\frac{L}{g}$,

$$v = V_1 - \frac{C' g}{L} \cdot \frac{t^2}{2} \dots\dots\dots\dots\dots\dots\dots\dots (5)$$

which gives v as a function of t at any instant.

Now, insert in Equation (2) the value of v from Equation (5), whence,

$$A \cdot d y = a (V_1 - V_2) \cdot d t - \frac{C' g a}{2 L} \cdot t^2 d t \dots\dots\dots (6)$$

where the variables y and t are separated.

The integration of Equation (6), with limits of y and zero for y, and t and zero for t, leads to,

$$y = \frac{a}{A} (V_1 - V_2) \quad t - \frac{C' g a}{6 A L} \cdot t^3 \dots\dots\dots\dots (7)$$

that is, y as a function of t.

We are now ready to find a relation between Y (that is, maximum y) and the corresponding time, T, as follows:

Substituting Y for y, and T for t, in Equation (7), we have, after re-arranging the terms,

$$Y = \left(V_1 - V_2 - \frac{C' g T^2}{2 L} \right) \frac{a T}{A} + \frac{C' a g T^3}{3 A L} \dots\dots\dots (8)$$

With V_2 for v, and T for t, in Equation (5), however, we find,

$$V_1 - V_2 - \frac{C' g T^2}{2 L} = \text{zero} \dots\dots\dots\dots\dots\dots (9)$$

Mr. Church.

and, consequently, Equation (8) becomes,

$$Y = \frac{C' \, a \, g \, T^3}{3 \, A \, L} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (10)$$

The constant, C', is now obtained by substituting p_2 for p, and T for t, in Equation (1), that is, $C' = \dfrac{p_2 - p_1}{T}$, which, in Equation (10), gives, finally,

$$Y = \frac{a \, g \, (p_2 - p_1) \, T^2}{3 \, A \, L} \ldots\ldots\ldots\ldots\ldots\ldots\ldots (11)$$

that is,

$$T = \sqrt{3} \sqrt{\frac{L \, A \, Y}{a \, g \, (p_2 - p_1)}} \ldots\ldots\ldots\ldots\ldots\ldots (R')$$

instead of the author's Formula (R), the latter differing from Formula (R'), however, only in having $\dfrac{\pi}{2}$ instead of $\sqrt{3}$; that is 1.5708 instead of 1.732, a discrepancy of about 10 per cent.

The writer was then led to seek the derivation of Formula (R) on other lines, by noting that the factor, $\dfrac{\pi}{2}$, was suggestive of a harmonic motion; and, accordingly, he started again with the assumption* that the motion of the water surface in the tank is harmonic, the initial point of this motion being the mid-point of a harmonic oscillation, so that the time from the start to the highest point would be,

$$T = \frac{\pi}{2} \sqrt{\frac{1}{C''}} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (12)$$

where C'' is the constant, the product of which by the distance (this product being taken with a negative sign), gives the acceleration, a', of the motion at any instant, namely,

$$a' = - \, C'' \, y \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (13)$$

Now $a' = \dfrac{d^2 \, y}{d \, t^2}$, an expression for which may be obtained by differentiating the two members of Equation (2) with respect to t; whence,

$$A \, . \, \frac{d^2 \, y}{d \, t^2} = a \, . \, \frac{d \, v}{d \, t} \ldots\ldots\ldots\ldots\ldots\ldots\ldots (14)$$

in which, if the value of $\dfrac{d \, v}{d \, t}$ from Equation (3) be inserted, there results.

$$a' = - \, \frac{a \, g}{A \, L} \, (p - p_1) \ldots\ldots\ldots\ldots\ldots\ldots (15$$

* This takes the place of Assumption 1.

Mr. Church.

or, for the highest point,

$$a'_2 = -\frac{a\ g}{A\ L}\,(p_2 - p_1) \qquad (16)$$

From Equation (13), however, we also have for the highest point,

$$a'_2 = -C''\ Y \qquad (17)$$

and the equating of these two values gives,

$$C'' = \frac{a\ g}{A\ L} \cdot \frac{(p_2 - p_1)}{Y} \qquad (18)$$

the insertion of which in the expression for T in Equation (12) leads to the author's Formula (R), namely,

$$T = \frac{\pi}{2}\sqrt{\frac{L\ A\ Y}{a\ g\ (p_2 - p_1)}} \qquad (R)$$

It is to be noted that the "harmonic" assumption just used, to take the place of the author's Assumption 1, simply consists in making $p - p_1$ proportional to the distance, y, instead of to the time, t; that is, in assuming the pressure in the tank to rise at a constant distance-rate, instead of at a constant time-rate. This is indicated by the fact that, from Equation (15), $p - p_1$ is proportional to the acceleration, a', of the motion of the surface of the water in the tank, and that, by Equation (13), this acceleration is assumed to be proportional to the distance, y (and of contrary sign), which is a characteristic property of harmonic motion.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

DISCUSSION ON FINAL REPORT OF THE SPECIAL COMMITTEE ON CONCRETE AND REINFORCED CONCRETE*

By Messrs. L. J. Mensch, G. S. Bergendahl, and E. S. Martin.

L. J. Mensch,† M. Am. Soc. C. E. (by letter).‡—The Final Report of the Special Committee on Concrete and Reinforced Concrete contains a great many excellent features, and, for its great devotion to the task allotted to it, the Committee deserves the thanks of the Society and of the general public, as all will benefit by its labors. It is to be regretted that sufficient funds were not at the command of the Committee so that it could have proceeded on the same scientific and practical lines as the French Committee in 1902 to 1905, or the German Committee from 1908 to 1915, or some other committees, the Special Committee of this Society on Steel Columns and Struts, for example. Mr. Mensch.

It seems that the Committee considered the tests made in various American colleges sufficient for the establishment of its rules. It nearly neglected the work of the French and German Committees, which solved many mooted questions. It was not unreasonable to expect that the Committee would have consulted these reports and would have made at least some new tests, in order to solve points left untouched by the others.

A great many of the Committee's rules do not contain any more information than may be found in the French report of 1905, and yet a great many more facts were known in 1916 than in 1905, and

* Continued from September, 1917, *Proceedings*.

† Chicago, Ill.

‡ Received by the Secretary, September 8th, 1917.

Mr. Mensch. would have permitted making more definite statements. Take, for example, the paragraph, "Freezing Weather." Tests could have been easily made, or reports from practice obtained, to give some more definite rules. It would have been very helpful to learn from the report how long to protect concrete in order to make it immune against freezing temperatures of various degrees.

In the chapter on "Forms", there is a very unfortunate paragraph, beginning, "Forms should be substantial and unyielding." All engineers know that lumber or steel deflects, and it would have been more proper to state the stresses and deflections for which forms are to be designed. The writer has never found that frozen concrete has a clear ring under the blow of the hammer, and the statement that this important test is unreliable ought to have been accompanied by the experimental facts.

The Committee recommends that, in columns, bars more than $\frac{3}{4}$ in. in diameter should be "properly squared and butted together in suitable sleeves." From the wording of this rule, one does not know whether to face the bars or merely hammer them square; neither would one know how long to make the sleeves. The fact is that it is practically impossible to obtain square ends or to place long bars on top of each other in order to bring their ends perfectly in contact; this, in practice, can only be obtained by a layer of cement which is poured into the sleeve before the upper bar is placed on the lower one, and tests show that a thin layer of cement grout is sufficient to produce a satisfactory transmission of stresses.

A great and important advance for American practice is the Committee's rule that the span length of continuous beams may be taken as the clear distance between faces of supports.

The assumptions recommended as a basis for calculation in Chapter VII are the same as those contained in the French report of 1905, and will hinder the clear understanding of the mechanics of reinforced concrete for another 10 years or more. The Committee states that it is well aware that these assumptions are not entirely borne out by experimental data, and that they are given in the interest of simplicity and uniformity. In Chapter X, working formulas based on these assumptions may be found, but engineers who are not very much in favor of concrete will not find these formulas sufficiently inviting for the adoption of reinforced concrete construction.

The writer is happy to state that the engineers of the building departments of our large cities have always been extremely courteous to him, and have never seen fit to compel him to undergo the ordeal of proving the safety of the structures he has submitted to them for approval by solving any formulas like those numbered (6), (11), (12), (16), (17), and (19). Formulas based on the conditions prevailing at the time of the ultimate load are considerably simpler and more

correct, because they agree with facts, while Formulas (6) to (19) agree with ancient theory. Mr. Mensch.

The writer would like to know the experimental facts on which the rules for the width of slabs in T-beams are based:

"(*a*) It shall not exceed one-fourth of the span length of the beam;
(*b*) Its overhanging width on either side of the web shall not exceed six times the thickness of the slab."

The tests of the French and German Committees show that one can count on a wider slab acting together with the stem.

Flat Slabs.—The Committee's ruling on bending moment coefficients is excellent, but the justification for it is rather lame. The Committee maintains with John R. Nichols, Assoc. M. Am. Soc. C. E., that the numerical sum of the positive and negative moments over the sections AF and GI of Fig. 2, disregarding the size of the column cap, is given by $\frac{W\,L}{8}$. The many tests which were made on buildings designed with bending moments with one-half this figure, or even less, compelled attention, and the Committee reduced this theoretical moment to an average of about $\frac{W\,L}{13}$ on account of the large size of column caps and because,

"Measurements of deformations in buildings under heavy load indicate the presence of considerable tensile resistance in the concrete, and the presence of this tensile resistance acts to decrease the intensity of the compressive stresses."

There is no reason whatever why, on these very same grounds, we should not allow a decrease of permissible moment coefficients for ordinary slabs, or slabs supported on four sides, or even T-beams with a small percentage of reinforcement. The statement that the presence of tensile resistance in the concrete acts to decrease the compressive stresses cannot be correct, because tensile stresses in the concrete mean less stress in the reinforcing; therefore, a reduction of the leverage of the centroid of tensile and compressive forces, and, for the same bending moment, will act to increase, and not to decrease, the compressive forces. If the Committee found by extensometer readings that the compressive strains are smaller than its theory warrants, then the proper deduction would have been that the theory was wrong and that the moments were actually smaller.

In his paper* Mr. John R. Nichols confessed that Grashof's formulas were a mystery to him, and maintained that from statics we are compelled to assume the numerical sum of positive and negative moments over one side and one center section as $\frac{W\,L}{8}$. The writer

* *Transactions,* Am. Soc. C. E., Vol. LXXVII, p. 1670.

Mr Mensch. would like to know whether the Committee thinks that the great geniuses, Poisson, Saint-Venant, Grashof, Föpple, and Love, did not know statics? In the writings of any of these men will be found the following rules for finding stresses in an elastic body:

(1) Every particle of the inside of the body must be in equilibrium.
(2) Every particle of the bounding surface must be in equilibrium.
(3) There must be a proper relation between stresses and strains.
(4) There must be a proper relation between strain and displacement.

Grashof derived his formula, $\frac{W\ L}{16}$, by establishing the fact that every particle of his deformed plate was in equilibrium, and, therefore, of course, the whole plate; but we cannot invert such a proposition by saying that, because the plate as a whole is in equilibrium, every particle of the plate must be in equilibrium; because Mr. Nichols and the Committee overlooked this important proposition, their formula is wrong. Professor H. E. J. Love clearly states in one of his papers:

"If by any intuition we obtain the stresses in an elastic body, they must fulfill the above conditions, and great errors were made by neglecting to check the results with above conditions."

It seems that nobody has yet had the intuition to prove in a simple way how the stresses in a flat slab can be ascertained, but the writer

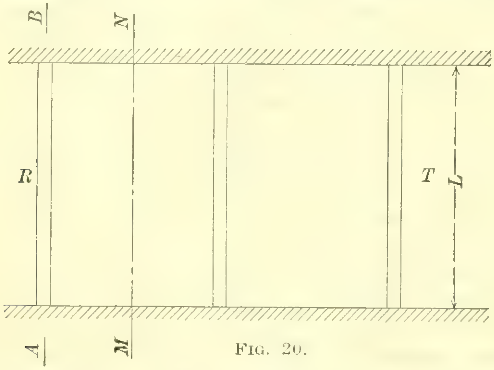

Fig. 20.

will endeavor to show by a few propositions the absurdity of the indiscriminate application of statics (which apply only to an absolutely rigid body or to a body affected by such small forces that the strains and stresses are practically nil, and in which it does not make any difference whether we make an error of 2 or more) to an elastic body.

(*1*).—Fig. 20 is a plan of a number of freely supported girders connected by slabs and uniformly loaded by w per sq. ft. According to statics, the moment in Section RT is $\frac{w L^2}{8}$, and the numerical sum

of the positive and negative moments at AB and MN equals $\frac{w\,D^2}{8}$, Mr. Mensch.
where D is the distance of the girders. Assume now that the slab between the girders is very thick in comparison with the girder; according to statics, the value, $\frac{w\,D^2}{8}$, is still in force, but if the slab is strong enough to carry the load in the direction of L, we can, without fear,

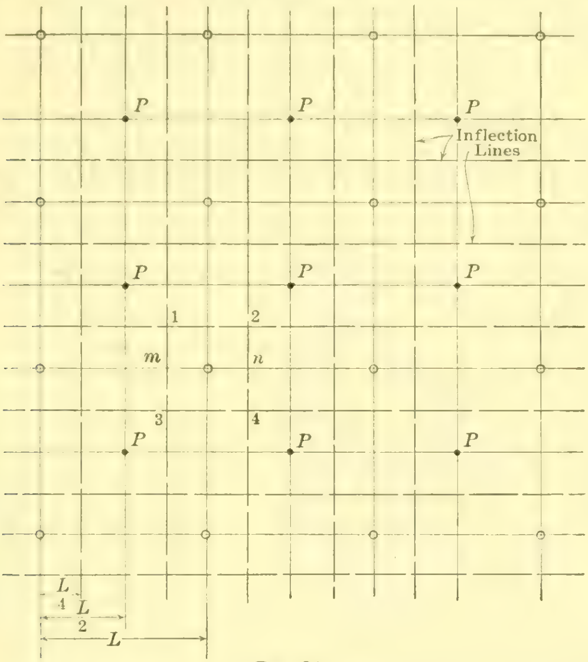

FIG. 21.

cut the slab at AB or MN, or in as many places in the direction of L as we please, and equilibrium will exist and the moments of $\frac{w\,D^2}{8}$ will vanish, even if D is very much longer or very much shorter than L. Hence we can make a static moment disappear without contravening any laws of Nature.

(2).—Take the example mentioned by Dr. Eddy in his discussion of flat slabs,* namely, a steel plate of infinite extent resting on rows of equidistant separate supports, arranged in corners of squares, and loaded in the center of the squares by equal concentrated loads, P. (Fig. 21.)

On account of the perfectly symmetrical manner of loading, the inflection lines must be half way between the supports and the panel

* *Proceedings*, Am. Soc. C. E., May, 1917.

Mr. Mensch.

centers. For example, a square, 1, 2, 3, 4, if cut out of the plate, may be considered to be affected by shears on all four sides, each equal $\frac{P}{4}$, and by a concentrated load, P, acting upward at the center. The shears at each side are not distributed uniformly, but, assuming for a moment that they are equally distributed, then, according to statics, the moment about mn would be $\frac{P}{4} \times \frac{L}{4} + 2 \times \frac{P}{8} \times \frac{1}{2} \times \frac{L}{4} = \frac{3}{32}\ P\ L$. If now we make the further assumption that the plate, 1, 2, 3, 4, consists of two plates, each of half the thickness of the original plate, and that on one plate are acting the shears, 1, 2 and 3, 4, and on the other one the shears, 1, 3 and 2, 4, the moment about a line through 0 for each plate becomes $\frac{P\ L}{16}$. Will anybody dare to say that by welding the plates together the moment $\frac{P\ L}{16}$ will become greater?

The same line of reasoning could be applied to a flat plate loaded with a uniform load. The lines of inflection in this case will be nearer the support, but again we will find a smaller moment about mn in the case of two plates than in that of one plate and, in order to obtain the moment of $\frac{W\ L}{8}$, we have had to make the assertion that two plates are stronger than one plate of twice the thickness.

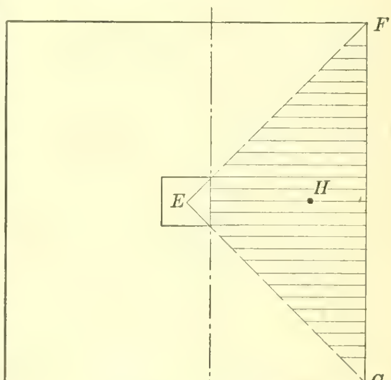

FIG. 22.

(3).—Fig. 22 is taken from *Bulletin No. 67* of the University of Illinois, "Reinforced Concrete Wall Footings and Column Footings", by A. N., Talbot, M. Am. Soc. C. E. This figure and the accompanying reasoning were evidently made after the results of all the tests were known, and Professor Talbot concluded that the best agreement with the tests can be obtained by assuming that one-fourth of the load (not one-half) is assumed to be acting in each direction. His intuition in this case was guided by a great number of tests; and the moment, derived from the assumption illustrated in Fig. 22, averaging $\frac{W\ L}{13}$, is exactly the same as that proposed by the Committee for flat slabs.

Mr. Mensch.

(*4*).—The Committee was extremely careful in most of its recommendations, and was always aware that any ruling it made must be made for average workmanship only; it was aware that only rarely is the designer the superintendent of construction (which ought to be the case), and if this conservative Committee allowed the sum of the moments in flat slabs to be taken as $\frac{W\ L}{13}$ instead of $\frac{W\ L}{6}$ in beams or slabs, the numerical sum of the theoretical moments at one side and one center of one panel, for this reason alone, must be $\frac{W\ L}{16}$ rather than $\frac{W\ L}{8}$.

(*5*).—It would have been more appropriate to point out the error in the books of Grashof, Föpple, or Lanza, than to lean on the tensile stresses of the concrete.

It is to be hoped that very soon proper tests will be made, proving that the so-called punching shear (which means shear near a support in a continuous structure) is not as dangerous as the ordinary shear in a simple beam, and that the ruling of the Committee allowing a shearing stress 25% higher than in a simple beam is correct.

The Committee advises that special attention be given to wall columns and corner columns, because the unequalized negative moment will be transmitted to the columns. Many engineers would have been very thankful if some rule had been given showing how to estimate such moments. The writer gave a very simple formula for these moments in his discussion* of Mr. Nichols' paper.

Floor Slabs Supported Along Four Sides.—The same liberal views shown in the case of ordinary flat slabs do not seem to have prevailed in the ruling on floors of this type. As previously mentioned, the tensile stresses in the concrete are just as effective in this case as in that of the flat slab on four columns, yet the Committee retained the ruling which originally was copied in the United States from German sources (where they did not believe in their own prophets), that a square slab shall be figured in the center according to $\frac{W\ L}{16}$. $\frac{W\ L}{20}$, or $\frac{W\ L}{24}$, according to whether the slab is a simple slab, partly or entirely fixed at the ends.

The ruling of the French Committee, based on tests of large slabs to destruction, gives the moments of $\frac{W\ L}{24}$, $\frac{W\ L}{30}$, and $\frac{W\ L}{36}$, L being the clear span of the slab. The Special Committee's meaning of the span—whether the clear span or the span from center to center of beams—is left indefinite in this case. The Committee evidently did

* *Transactions*, Am. Soc. C. E., Vol. LXXVII, p. 1682.

Mr. Mensch. not consult the tests to destruction on such slabs made by Professor Bach on behalf of the German Committee and others, or it would not have limited their use to cases where the length did not exceed one and one-half times the width. There are a number of tests on record where slabs of a length of two and one-half times the width showed an immense increase of strength over a slab supported only on the long sides.

The Committee's ruling is in full accordance with the building codes of nearly all large cities in the United States, and this unfavorable ruling tempts many designers to use flat slabs where they should not be used. The writer wishes to refer to concrete buildings from two to five stories in height, where, by the use of flat slabs, the stability of brick walls is endangered, or where, instead of the more economical brick walls, a skeleton construction is used in order to take care of the negative moments of the flat slabs in the outside panels. A more liberal ruling for slabs supported on four sides is urgently needed.

The chapters on diagonal tension, shear, and bond do not lay enough emphasis on the great importance of hooks at the ends of bars. Too much reliance is placed on the now nearly general use of deformed bars. The German Committee had tests made on about 100 beams, all of the same size, reinforced with the same percentage of steel, and all designed for the special purpose of finding the influence of hooks when only straight bars were used or when one-half or more of the bars were bent up, and in parallel series with and without stirrups. In this long series, not one beam can be cited where the omission of hooks did not show a marked reduction of the ultimate load over bars with large hooks. Where hooks were omitted in one-quarter of the bars, the drop was about 10%, where one-half of the bars were without hooks, the drop was about 20%, and where all the bars were without hooks, the drop was 50% and more. This importance of hooks in simple beams is greatly under-estimated in the United States, and wrong conclusions are often drawn from tests, because the great influence of hooks is overlooked.

Professor Moersch made tests on beams reinforced with deformed bars without hooks, and found the drop the same as with plain bars without hooks. The great importance of hooks is also shown in the tests made by Professor Talbot on wall and column footings. In the face of these facts, the paragraph: "As an additional safeguard, end anchorage may properly be used in special cases" and "anchorage of longitudinal bars at the ends of beams is advantageous," will not impress on the reader the importance of the subject.

It probably is a surprise to many engineers that the Committee advises the use of only two-thirds of the external shear in making calculations for the web reinforcing, yet this ruling is good engineering, and may be accounted for by the following consideration: A concrete

beam has a considerable arch action, and may be considered to be a combination of truss systems like those shown in Figs. 23, 24, 25, and 26. Mr. Mensch.

In a plain concrete beam, the truss system, *A*, will carry the same load as System *B*, for the reason that the width of the bands, *a* and *b*, is probably very much greater than in the center of the beam, hence the stresses are very low and the strains in compression are the same as in tension.

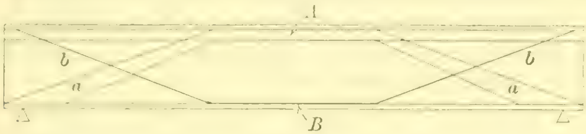

FIG. 23.

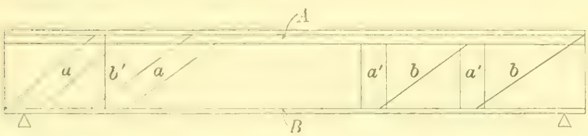

FIG. 24.

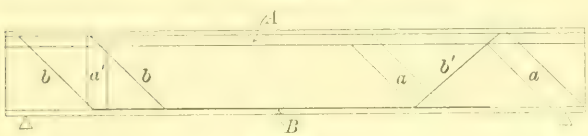

FIG. 25.

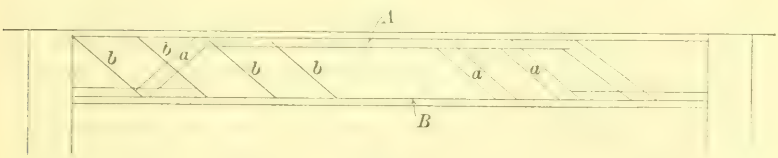

FIG. 26.

In a concrete beam with a low percentage of straight bars only, without stirrups (Fig. 23), the truss System *A* will carry a great deal more than System *B*, because the stresses in *a* and *b* are now very much higher, and *b* in tension is weaker than *a* in compression. In order that System *A* shall be effective, there must be a good connection between *a* and the horizontal reinforcement. This connection is generally furnished by the bond of the concrete to the steel rods, and when this bond is partly overcome, System *B* will take up a larger load until the stresses in *b* cause the concrete to crack, when System *A* will gradually take up the entire load.

At this time, practically the entire horizontal component of the force in *a* must be taken up by the reinforcement near the support.

Mr. Mensch. The vertical component of a will have a tendency to bend the reinforcement, even to push it out of the concrete, and to strain the concrete in tension in a horizontal direction. In fact, many tests show horizontal cracks near the steel rods, which cracks will weaken the bond, and the connection between a and the steel rods will have to rely on the bond beyond the supports. Only rarely are freely supported girders extended out a sufficient length beyond the supports to form this connection, and the importance of hooks at the ends of the bars may now be seen clearly. Some of the tests of the German Committee seem to indicate that a large round hook in a 1-in. round bar is sufficient for an anchorage of 13 000 lb.

Where the shear in a concrete beam is low, the trusses will have a shape like that shown in Fig. 24. In this case, the diagonals, a and b are much steeper, and will require less bond than those in Fig. 23. The truss systems will be in action as long as the stresses, b and b', are below the ultimate tensile strength of the concrete.

Beams are generally tested with concentrated loads, and vertical cracks in the concrete will appear near the points of application of the loads, which will diminish the effective area for the diagonals, b, and the diagonal cracks will generally start at the end of a vertical crack. This behavior of concrete beams, reinforced with straight rods, and with a low end shear, is beautifully illustrated in the photographs, 22 to 28, *Technologic Paper No. 2,* of the U. S. Bureau of Standards.

When stirrups are used, the action of b and b' is not limited any more to the tensile strength of the concrete alone, but the action of b' is limited by the connection between a and b'. The stirrups partly take up the stresses in b' and, by components, the stresses in b. Hooks at the ends of stirrups, when the latter are highly strained, are just as important as in the case of horizontal bars, and a great number of small stirrups will be more effective than a few large stirrups. Systems A and B can be replaced by a number of systems, in order to make all stirrups effective, but the final result can be better understood by assuming only one System A and one System B as acting. Because the horizontal component of a in Fig. 24 is smaller than that in Fig. 23, the bond stresses will be smaller, but, as a rule, even a great many stirrups will not suffice to allow the omission of the hooks at the ends of the horizontal bars.

Where bent-up bars are used, the main systems, A and B, will have about the shape shown in Fig. 25. The forces in the diagonals will be limited again by the bond at the upper ends of the bent-up bars just as a is limited by the bond at the ends of the horizontal bars. It may be noticed that the second bent-up bar from the support is in common for both systems A and B, which explains the fact that diagonal cracks

generally appear first quite a distance from the support. Stirrups will be helpful, especially in the part where the bent-up bars are omitted, or near the supports, in order to prevent the weakening of the bond of the horizontal bars. In a well-designed beam, truss systems A and B will be equally effective, and each system will take up half the shear, but, evidently on account of the indeterminate action of the trusses, the Committee decided on the rule to use two-thirds of the external shear for the calculation of shear members. That a graphical solution of stresses in shear members will be more correct than Formulas (22) to (27), is evident from the foregoing. Inasmuch as hooks play such a large rôle in the effectiveness of reinforcement, proper tests in this direction ought to be made at an early opportunity. Mr. Mensch.

In Chapter VIII, the writer finds a great advance in the ruling to allow 35% of the compressive strength in bearing, where the surface of the concrete is twice the loaded area.

The column ruling, although liberal and in full agreement with tests, is not broad enough for all practical applications. The Committee should not have placed the lower limit of spiral reinforcement as 1%; it should have been placed at ½%, and it should have allowed a higher upper limit, and given proper working formulas for these cases.

The writer is greatly surprised that this conservative Committee allows a stress of 16 000 lb. per sq. in. on mild reinforcing, without specifying that it should apply only to properly twisted bars, or otherwise to state, like the French Committee, that this ruling is on account of the demand of practice (high-carbon steel bars are not a commercial article in France), and that it really results in a factor of safety of considerably less than 3 with the best of workmanship. No mention is made of the use of high-carbon steel bars, and it can be stated positively that the commercial high-carbon steel bars (even re-rolled bars) can be stressed to 26 000 lb. per sq. in. to give the same strength in a reinforced concrete beam as mild steel bars stressed to 16 000 lb. per sq. in.

No ruling which the Committee has made is so discordant with general practice and will be so generally disregarded as the deliberate omission of high-carbon steel bars, and it positively damages the industry. The writer is well aware that structural steel designers with little experience in reinforced concrete are all in favor of the Committee's ruling.

The ruling in regard to the various values of n in Paragraph 8, Chapter VIII, only confuses the student. It was originally proposed by the French Committee for the use of column reinforcing, but it would have been very much simpler to recommend to calculate with an ultimate resistance of the steel rods of, say, 30 000 lb. per sq. in., which is, in fact, all that the various values of n result in.

Mr. Bergendahl. G. S. BERGENDAHL,* M. AM. SOC. C. E. (by letter).†—In the discussion‡ of Professor Eddy's discussion§ by F. E. Turneaure, M. Am. Soc. C. E., it appears that Mr. Turneaure has not properly treated the shear in a flat plate floor. For example, in Fig. 12, he treats the shear on the column faces, *C-B* and *D-E,* as entering into the moment about *J-K.* Moment is a directed quantity, and shear enters into the production of moment only as it is normal thereto. When parallel thereto, it does not enter into the moment under consideration. Therefore, Mr. Turneaure has incorrectly taken twice the shear that actually operates in producing a moment normal to the line, *J-K,* and is in error in his criticism of Professor Eddy for that reason. This is elementary. Professor Talbot likewise appears to misunderstand this simple relation.

The Committee has correctly recognized this principle in its treatment of the square slab supported on beams on four sides in its division of the moment between the two directions, and it seems only thoughtlessness to have failed to follow the same correct principle in treating the shears on planes at right angles to each other, when the slab is supported on columns at the four corners instead of on beams at the four sides.

The report requires twice as much steel as that necessary to develop the strength of the concrete in flat slab construction. Therefore, 50% of the steel which the Committee's rule requires to be embedded, is wasted. It is strange that, with all the facilities this Committee has had at its disposal, it has not determined the relation of the quantity of steel that is required to balance and develop the full strength of the concrete in this type of construction. Any information which furnishes grounds for rational economy in engineering structures should, as Professor Talbot says, be welcome, even if it requires the frank admission of erroneous assumptions on the part of some of our members.

After building and testing flat slab floors for the past nine years, the writer has some very definite and exact information as to the deportment of structures of this kind in regard to which it would seem the Committee was entirely lacking, in view of the nature of its report. The advantages of this kind of construction can only be secured by the relatively close spacing of the slab rods. Those who have failed to carry out their construction with due consideration of this feature have secured results in keeping with beam action rather than the advantages of imitation of plate action.

The thing which is most striking about the Eddy theory is that by it one can, in a properly designed flat slab, actually predict and

* Chicago, Ill.

† Received by the Secretary, September 13th, 1917.

‡ *Proceedings,* Am. Soc. C. E., August, 1917.

§ *Proceedings,* Am. Soc. C. E., May, 1917.

compute in advance the deflections that will be measured under load and the stresses that will be found in the steel. Substantial agreement of theory with ascertained fact may appear of no weight to Professor Talbot and certain members of the Joint Committee, but it is regarded by the practical engineer as the only criterion on which sound judgment can safely be founded. Mr. Bergendahl.

E. S. Martin,* Assoc. M. Am. Soc. C. E. (by letter).†—There are some recommendations in the report which merit critical consideration. Mr. Martin.

Under the head of "Floor Slabs Supported Along Four Sides", it provides that the reinforcement over the outer quarter slab widths may be one-half of that in the middle belt. This is a distribution which the writer has used many times, and believes to be conservative and well founded, but he fails to see any reason for using more reinforcement in the middle belt when the outer belts are reduced than when uniform reinforcement is provided. Certainly, the relief from flexure afforded to the slab along the supporting beams by these beams does not add to the bending moment sustained by the middle portion of the slab.

There is a requirement under Section 6 of Chapter VII, that reinforcement at the supports of continuous beams should extend beyond the inflection points a sufficient distance "to develop the requisite bond strength." This sentence, it is believed, will not generally be interpreted to mean exactly what it says, but, on the contrary, will be considered to require that laps of reinforced rods shall be sufficient to develop their full tensile strength at the point of inflection, notwithstanding that the tensile stress is zero at this point. Nearly all cases met in general building work are properly taken care of when the steel extends to the inflection points. For stubby, heavy beams, in which the tensile stress of the continuous steel piles up faster than the allowed bond provides for, the writer would prefer using a hooked end for anchorage rather than depend on tension being developed in the rod where it is surrounded by concrete in compression.

For "flat slabs", it is recommended that the capital slope be not more than 45° with the vertical; also, that the depression be four-tenths of the span in width and not more than one-half as thick as the slab. In a few cases the writer has designed floors with wide flat capitals, thinking that he had provided both capital and depression in one form. In this design he has assumed that the capital extended to the point where the total concrete thickness was twice the slab thickness. Although this design does not meet the requirements of the report, the writer is unconvinced that it is not good engineering practice—that it is not as good as that recommended by the Committee. The Committee's requirements are in danger of being followed blindly by

* New York City.

† Received by the Secretary, September 27th, 1917.

Mr. Martin. those without independent judgment, and should not contain unnecessary provisions limiting freedom of design.

The method of slab design recommended—by inner and outer belts—is not the best that can be devised, although it has been used with different formulas in both the Philadelphia and Chicago rulings. The objection to this method is the fact that the critical moment and stresses occur around the capital and decrease outward therefrom. To make the belt method at all workable, it is necessary to limit its application to designs having a fixed minimum ratio of capital diameter to span; otherwise one may find himself relying on a belt of cantilever steel several times as wide as the capital, in which only the middle rods are really effective. A minimum limiting size of capital interferes with freedom of design, and unnecessarily so, because good designs, without over-stressing, may be made without capitals, if the slab is thick enough and contains sufficient steel.

The writer has made a comparison of the results found by applying the Committee's method to several buildings on which careful tests have been conducted. Detailed reports of these tests have been published in engineering journals.* For purposes of comparison, it is also of interest—and what is more, necessary—to examine the results of similar tests on beams, and beam and slab buildings. These figures are given in Tables 6 and 7. The flat slab figures are given in Table 5. Although the writer has not presented as many beam data as desirable, those given are sufficient for general conclusions.

In studying these figures, one should keep in mind the vital differences between the beam and slab floor and the flat slab floor. A glance at the computed and measured stresses for the increasing loads of Beam 72 (Table 6) shows that the tensile value of the concrete keeps the steel tension below the computed values for light loads, and that the concrete effect dies out with the heavy loads, loads causing measured stresses of from 16 000 to 20 000 lb. of unit tension in the steel. The percentage of tension steel (1.27) in this case is high. A lower percentage would have shown greater differences between computed and measured stresses. One can form no conclusions from the measured stress when this is low, but when it is high—from 16 000 to 20 000 lb.—the elongation is such that the concrete is broken up and its tensile effect destroyed. The percentage of reinforcement is not of much importance in such a case.

Now, in the beam and slab floors, it is evident that the entire slab is effective in tension at the supports of the beams and girders, and, consequently, the percentage of reinforcement (ratio of tensile steel to tensile concrete) is low, though the contrary is true at the mid-span of a beam. Such conditions do not exist in the flat slab, and allowances should be made. For instance, at the column capital of a flat

* Most of them are in the *Bulletins* of the University of Illinois.

Mr. Martin.

TABLE 5.

Building.	Panel and test.	Steel and slab.	MEASURED. Mid-span.	MEASURED. Capital.	AMERICAN SOCIETY. Mid-span.	AMERICAN SOCIETY. Capital.	CHICAGO. Mid-span.	CHICAGO. Capital.
Northwestern Glass Company, Minneapolis.	16 ft. by 17 ft. 400 lb.	8 in.; 4 ft. 6-in. capital.	f_s 8 200	f_s 22 000	....	....	....	....
		15 ⅜-in. rods, 7 ft. wide.	4 300	20 000	25 800	21 500	25 800	28 800
		8 1⅛-in. rods, radial.		17 500	20 900	....	14 400	....
			Wall 12 000	f_c 1 200	....	....	....	....
			panel 15 000	900	....	880	....	1 110
Deere and Webber, Minneapolis.	18 ft. 8 in. by 19 ft. 1 in. 340 lb.	9 3/16 in.; 4-ft. capital.	f_s 6 900	f_s 20 700	26 600	19 500	26 400	27 700
		14 7/16-in. rods, diag.	5 100	18 300	19 500	....	19 000	....
		12 7/16-in. rods, direct.						
		7 ft. 3 in., wide.	....	f_c 795	....	750	....	990
		8 1¼-in. rods, radial.	....	750	....	....	....	....
Larkin Company, Chicago.	20 ft. by 24 ft. 2 in. 370 lb.	9 in.; 5-ft. capital.	f_s 3 300	f_s 3 500	26 200	....	25 000	18 100
		8 ft., drop 6¾ in.	3 000	2 500	19 000	15 600	15 100	....
		19 ½-in. rods, diag.						
		23 ½-in. rods, long.						
		13 ½-in. rods, short.		f_c	....	565	....	....
		8 ⅞-in. rods, radial.	f_c 550	570	....	....	....	490
J. Franks, Chicago.	19 ft. 4 in. by 20 ft. 3 in. 250 lb.	9 ¼ in.; 3 ft. 8-in. capital, drop 4 in.	f_s 4 540	f_s 4 575	18 800	7 300	12 700	8 050
		18 ½-in. rods, diag., lapped.	1 070	3 440	8 000	....	7 500	....
		16 ½-in. rods, direct.	....	f_c	....	310	....	....
		9 ft. 6 in. wide.	....	677	....	....	....	335

Mr. Martin.

TABLE 5.—(*Continued*)

Building.	Panel and test.	Steel and slab.	Measured. Mid-span.	Measured. Capital.	American Society. Mid-span.	American Society. Capital.	Chicago. Mid-span.	Chicago. Capital.
Shredded Wheat Company, Niagara Falls.	20 ft. by 22 ft. 191 lb. 25-in. columns.	7 ½ in.; 3 ft. 6-in. capital. Drop, 8 ft. 6 in. by 2 in. Long, 10 ⅝-in. rods + 8 ½-in. rods. Long cant., 17 ⅝-in. rods + 8 ½-in. rods. Short, 10 ⅝-in. rods + 8 ½-in. rods. Short cant., 15 ⅝-in. rods + 8 ½-in. rods. Long mid., 12 ½-in. rods. Short mid., 10 ½-in. rods.	f_s { 16 000 20 000 Wall panel { 11 000 20 000 f_c { 1 400 } Int. 900 } wall	f_s 16 000 ... ... f_c 1 400	15 900 25 300 540	18 100 690	... ...	... ...
Central Terminal Railway, Chicago.	24 ft. by 24 ft. 700 lb. 32-in. columns.	18 in. and 22 in.; 5 ft. 6-in capital. Drop, 9 ft. 0 in. by 12 in. 20 ¾-in. rods, direct and diagonal, and over columns.	f_s { 4 800 5 000 f_c { 720 600	f_s 3 600 ... ... f_c 600	11 600 7 000 ... 300	9 000 300	... ...	... ...
Schulze Bakery Company.	17 ft. 6 in. by 20 ft. 320 and 720 lb. 26- and 28-in. columns.	9 in. 4½-ft. capital. Drop. 7½ ft. by 5 in. 20 $\frac{7}{16}$-in. rods, diag. 23 " " " long. 17 " " " short. Cant. diag., doubled + ⅔ direct belts	f_s { 5 000 2 500 f_c 450 f_s { 10 000 6 500 f_c 900	f_s 9 000 f_c 300 ... f_s 10 000 ... f_c 840	21 500 13 200 600 48 400 29 700 1 350	12 700 620 ... 28 500 ... 1 400 ...	... ...	... ...
Curtis Leger Fixture Company.	17 ft. 10 in. by 19 ft. 200 and 400 lb.	8 in.; sl. cap. 4 ft. 6 in. 8 ½-in. sq. diag. 9 " " " short. 10 " " long. 10 " " square, all ways, wall.	f_s { 3 500 7 000 f_c { 13 000 7 000	f_s 4 000 f_c 400 f_s 12 000 f_c 900	15 400 11 200 30 800 22 400	10 200 500 20 400 1 000	... ...	... ...

Mr. Martin.

TABLE 6.

From *Bulletin 28,* University of Illinois, October 5th, 1908, Page 16.

Beam 72. Percentage = 1.27

Measured.	Computed.	Measured.	Computed.	Measured.	Computed.
1 200	6 300	14 700	17 400	29 400	29 300
2 100	8 000	17 700	19 300	31 800	31 400
3 600	9 700	20 100	21 400	36 900	35 300
6 000	11 500	23 100	23 300	42 000	39 300
7 500	13 400	25 800	25 300	47 700	42 900
10 500	15 300	27 900	27 300	61 200	47 600
				108 000	49 600

TABLE 7.

			Wenalden Building.		Turner Carter Building.	
			Stresses, 400 lb. test.		Stresses, 300 lb. test.	
			Measured.	Computed.	Measured.	Computed.
Girder.	End.	Steel	13 000	44 000	......	31 000
		Concrete	2 200	1 700	900	1 200
	Mid-span.	Steel	17 000	19 000	9 300	12 500
		Concrete	......	420	200	300
Intermediate beam	End.	Steel	16 000	36 000	7 800	21 500
		Concrete	2 000	1 900	1 200	1 300
	Mid-span.	Steel	16 000	22 000	9 000 (?)	18 500
		Concrete	......	440	480 (?)	380
Column beam.	End.	Steel	11 000	69 000	......	19 600
		Concrete	......	......	1 250	1 200
	Mid-span	Steel	15 000	26 000	10 600	17 000
		Concrete	......	......	380	350
			12 months old.		50 days old.	

Computed stresses: $\frac{W K}{12}$; L = clear span + 3 in.

slab floor, the maximum quantity of steel is used, and the only concrete in tension is that above the neutral axis of the slab for the circumference of the capital. The conditions as regards the tensile value of the concrete are similar to those at the mid-span of a beam.

Table 5 shows that the Committee method allows a liberal margin

Mr. Martin. in all cases over the measured stresses in the reinforcement, but no margin whatever for the concrete stress at the capital, although the latter is measured on an 8-in. gauge line, beginning at the capital, and is based on a nominal value for the modulus of elasticity. In the writer's opinion, the depression thickness, in general, should be not less than one-half the slab thickness rather than not more than this, as a limit which the Committee states.

Comparison of measured and computed stresses in the beam and slab floors shows evident arching, as the concrete stresses at the ends check roughly, and the steel stress at mid-span measures less than that computed, even at the higher values, and the concrete in tension is of too small a quantity to have much effect.

Several discussions on the report of the Committee have already been published in the *Proceedings* of the Society, but these have had reference to the theoretical phase. Although the writer has much respect for many of the theories put forth to explain the deportment of a flat slab, he does not consider that any of them so far advanced—not even the Committee's assumptions—have been established beyond question, so that they may be confidently followed in quantitative design, that is, to give definite values of stresses. He feels much safer when he knows that stresses are conservative, in the light of all the measured stresses of the numerous careful tests that have been conducted.

The writer questions the necessity of providing a wide margin of computed stress over the highest measured stress at the capital edge. The standard beam designs do not require a material margin at mid-span where corresponding conditions exist. He questions the necessity of providing a margin of from 300 to 500% at mid-span, except when the measured stresses are low, with low percentage. He questions the adequacy of the 20% additional for outside panels, if the interior panels are economically designed. Test measurements show that wall panels have stresses from 50 to 100% greater than interior panels, depending on the span length.

MEMOIRS OF DECEASED MEMBERS

NOTE.—Memoirs will be reproduced in the volumes of *Transactions*. Any information which will amplify the records as here printed, or correct any errors, should be forwarded to the Secretary prior to the final publication.

HAROLD PARKER, M. Am. Soc. C. E.*

DIED NOVEMBER 29TH, 1916.

Harold Parker, the son of George Alanson and Harriet Newhall Felton Parker, was born in Charlestown, Mass., on June 17th, 1854. He was a direct descendant of Captain Joseph Parker, who settled in Newbury in 1638, and of Nathaniel Felton, who settled in Salem in 1633.

After receiving a preliminary common school education in Philadelphia, Pa., and Lancaster, Mass., Mr. Parker entered Phillips Exeter Academy, from which he was graduated. He then entered Harvard College, where he continued his studies during 1871 and 1872.

Leaving college for a business career, he was first employed in the capacity of Foreman with the Pennsylvania Steel Company. In 1874, he returned to his home town, Lancaster, Mass., and became a member of the civil engineering firm of Parker and Bateman, continuing as such until his death.

Notwithstanding the fact that he was of modest disposition and not of the office-seeking type, Mr. Parker was prominent in the political affairs of Lancaster, and was twice elected to represent his district in the State Legislature. His best years in public service, however, were devoted to the work of building up the highway system of his native State. In 1900 he was appointed by the Governor as a member of the Massachusetts Highway Commission, and on the resignation of William E. McClintock, M. Am. Soc. C. E., in June, 1908, he was made Chairman, which position he held until 1911, when he resigned again to devote his time to other than public office. His ability and tireless industry added to the good repute for gaining progress and results for which this Commission had already become widely known. The revolution in methods of highway transportation and consequent revolution in methods of highway construction was at its height during Mr. Parker's administration as Chairman of the Commission, and many who are continuing the work so ably carried on by him, are indebted to him for his thought and foresight.

Immediately following his resignation as Highway Commissioner, he became Vice-President of a large contracting firm, thus continuing to devote his time to road and pavement work.

* Memoir prepared by A. W. Dean, M. Am. Soc. C. E.

In 1913 and 1914, Mr. Parker served the State of New York in an advisory capacity, being designated by Governor Dix as a member of the Advisory Board of the Highway Department.

Being a lover of nature, he spent his leisure in the fields and woods, taking much interest in the forests and natural resources of his neighborhood and State, and considering as a pleasure rather than a duty his services as Chairman of the Massachusetts Forestry Commission and of the Wachusett Mountain Reservation Commission.

Indication of Mr. Parker's recognition by others in his chosen field of effort is shown by his selection as President of the Massachusetts Highway Association and of the American Road Builders' Association, in both of which societies he took great interest.

Mr. Parker was a man of the sturdy New England type, of high principles, and a most true friend to those who won his friendship.

He is survived by his widow, Elizabeth B. (Bartol) Parker (to whom he was married in 1884), and by one son and two daughters.

Mr. Parker was elected a Member of the American Society of Civil Engineers on June 7th, 1899.

PAPERS IN THIS NUMBER

"**PULSATIONS IN PIPE LINES, AS SHOWN BY SOME RECENT TESTS.**" H. C. VENSANO. (To be presented Nov. 7th, 1917.)

"**A BRIEF REVIEW OF TRIGONOMETRICAL MATHEMATICAL TABLES, AND A CONTEMPLATION OF THE SPECIFICATIONS FOR TRIGONOMETRICAL TABLES FOR GENERAL USE.**" VIRGIL A. EBERLY.

"**THE HELL GATE ARCH BRIDGE AND APPROACHES OF THE NEW YORK CONNECTING RAILROAD OVER THE EAST RIVER IN NEW YORK CITY.**" O. H. AMMANN. (To be presented Nov. 21st, 1917.)

"**STRESS MEASUREMENTS ON THE HELL GATE ARCH BRIDGE.**" D. B. STEINMAN. (To be presented Nov. 21st, 1917.)

PAPERS AND DISCUSSIONS CURRENT IN PROCEEDINGS

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AMERICAN SOCIETY

OF

CIVIL ENGINEERS

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MINUTES OF MEETINGS

OF THE SOCIETY

October 17th, 1917.—The meeting was called to order at 8.30 P. M.; Director L. D. Rights in the Chair; Chas. Warren Hunt, Secretary; and present, also, 127 members and 9 guests.

A paper by the late H. M. Chittenden, M. Am. Soc. C. E., entitled, "Detention Reservoirs with Spillway Outlets as an Agency in Flood Control," was presented by the Secretary, who also read communications on the subject from Messrs. Arthur E. Morgan and Alexander Rice McKim. The paper was discussed orally by T. Kennard Thomson, M. Am. Soc. C. E.

Charles Evan Fowler, M. Am. Soc. C. E., addressed the meeting in reference to the late Gen. Chittenden, with whom he was intimately

acquainted, giving an appreciation of his qualities as a man and as an engineer.

A paper by A. L. Sonderegger, M. Am. Soc. C. E., entitled "Hydraulic Phenomena and the Effect of Spreading of Flood Water in the San Bernardino Basin, Southern California", was presented by the Secretary, who also announced that this paper had been presented before the Southern California Association of Members of the American Society of Civil Engineers at its meeting of April 11th, 1917. Discussions by Messrs. William Mulholland and Charles H. Lee, presented at that meeting, were read by the Secretary.

The Secretary announced the election of the following candidates on October 9th, 1917:

As Members

James Allen, Olympia, Wash.
Charles Erwin Barglebaugh, El Paso, Tex.
John Bealle Battle, El Paso, Tex.
Francis Marion Butler, Orbisonia, Pa.
Le Roy Francis Harza, Sault Ste. Marie, Ont., Canada
Herbert Henderson, Port Arthur, Tex.
Nils Herman Baashuus Jessen, New York City

As Associate Members

Henry Stanley Allen, Kansas City, Mo.
Gustaf Birger Andreen, Elmhurst, N. Y.
Charles Newton Bainbridge, Lombard, Ill.
Eugene Duston Barstow, Cuyahoga Falls, Ohio
Fred Bennett, Denison, Tex.
Odin Baltimore Bestor, Albany, N. Y.
Kitchell Monckton Boorman, New York City
Horace Trowbridge Brown, San Francisco, Cal.
Oscar Ernest Bulkeley, Jackson, Mich.
Paul Nels Carlson, Seattle, Wash.
Merwin Bishop Carson, Honolulu, Hawaii
Charles Grover Chamberlin, Wichita, Kans.
Hugh William Crawford, Topeka, Kans.
Horace Culpeper, Houston, Tex.
Bert E. Dodge, Madison, Wis.
Claude Lacy Douthett, Sidell, Ill.
George Lucas Dresser, New York City
Harold Mansfield Eddy, Canton, Ohio
Rily Ellsworth Elgen, Chattanooga, Tenn.
James Robert Fairman, Kansas City, Mo.
George Keithley Farner, San Francisco, Cal.
Arthur Virdin Foard, Baltimore, Md.
Nathan Henry Gellert, Wayne, Pa.
George Christian Graeter, Sullivan, Ind.

Guy Morley Harbert, Clarksburg, W. Va.
Frederic George Healy, Bridgeport, Conn.
Charles Henry Holstlaw, West Palm Beach, Fla.
Frank Johnson Householder, Baltimore, Md.
Bernhard Faaborg Jakobsen, San Francisco, Cal.
Albert Frederick Johntz, Camaguey, Cuba
William Donelson Jones, Los Angeles, Cal.
Benedict Joseph Kaiser, Pittsburgh, Pa.
Frank Hiram Knapp, Blasdell, N. Y.
Henry Way Leal, Chicago, Ill.
William Shannon Lohr, Lancaster, Pa.
Hugh John Lumsden, Kansas City, Mo.
Harry England McCool, Duluth, Minn.
William Frederick McDonald, Milwaukee, Wis.
William Henry Mead, Houston, Tex.
Melville Sampson Miller, Brooklyn, N. Y.
Kendall Tuttle Murphy, Baltimore, Md.
James Lee Murray, Mittineague, Mass.
Alva Harold Perkins, Am. Exp. Force, France
Frank Hurd Pickett, Bridgeport, Conn.
James Handyside Reding, Fort Sheridan, Ill.
Harlow Loveridge Rockwell, Buffalo, N. Y.
George Viall Salle, Cincinnati, Ohio
Charles William Sylverius Sammelman, St. Louis, Mo.
Beale Melancthon Schmucker, Haddon Heights, N. J.
Ivan Oscar Shaffer, Summit, N. J.
Kenneth Shibley, San Francisco, Cal.
Robert Earle Shipley, Texarkana, Tex.
Samuel Spaulding Stevens, Washington, D. C.
Herbert Miller Stofflet, Chicago, Ill.
Edward Stuart, Harrisburg, Pa.
John Abraham Van Den Broek, Ann Arbor, Mich.
Julius Verner, Linden, N. J.
Joseph Johnson Vogdes, Philadelphia, Pa.
Harrison Billingsley Walton, Little Rock, Ark.
Wylie Brodbeck Wendt, Manhattan, Kans.
Herman Frederick Wiedeman, Atlanta, Ga.

As Juniors

George Robert Bickel, Louisville, Ky.
Harold Follmer Bucher, Watsontown, Pa.
Merton Shumway Fogerty, Groveton, N. H.
Walter Fritz, New York City
Marion Columbus Huckaby, Baton Rouge, La.
William Francis Lockhardt, Brooklyn, N. Y.

William Maxwell Marker, Stanford University, Cal.
Albert Clark Matthews, Jr., Orange, Va.
Mortimer Louis Neinken, Brooklyn, N. Y.
James Parker, Tulsa, Okla.
Alfred Renshaw, New York City
Herman Ritow, New York City
Norman Fraser Strachan, Lawrence, Kans.
José Alexandre Teixeira De Mello, Troy, N. Y.
Stephen Riggs Truesdell, Jackson, Mich.
Clifton L'Orignal Weekes, Cincinnati, Ohio
Lee Hoomes Williamson, Rancagua, Chile

The Secretary announced the transfer of the following candidates on October 9th, 1917:

From Associate Member to Member

George Ellsworth Barrows, Buffalo, N. Y.
Herbert Ernest Bellamy, Crib Point, Victoria, Australia
Robert Mavin Cooksey, Baltimore, Md.
Walter Louis Du Moulin, Antofagasta, Chile
Augustus Griffin, Manteca, Cal.
Victor Frank Hammel, New York City
Charles Frank Healey, Chicago, Ill.
Reuben Sylvester Peotter, Paramaribo, Dutch Guiana
Bion Harman Piepmeier, Springfield, Ill.
Paul Leon Pierce, New York City
George Moses Purver, Brooklyn, N. Y.
Leroy Norman Reeve, Denver, Colo.
Robert Bruce Tinsley, Chuquicamata, Chile

From Associate to Associate Member

Clark Edwin Mickey, Lincoln, Nebr.

From Junior to Associate Member

Franklin Rea Allen, Pine Bluff, Ark.
Russell Vincent Banta, New York City
Tom Allen Bither, Berkeley, Cal.
Sidney Breese Bowne, Mineola, N. Y.
Arthur Gray Butler, Cleveland, Ohio
Harry Foster Ferguson, Urbana, Ill.
James Gordon Goodfellow, Edinburgh, Scotland
Homer More Hadley, Vancouver, B. C., Canada
Lewis Merrick Hammond, Washington, D. C.
Allen Hoar, Alameda, Cal.
Leigh Patterson Jerrard, Madison, Wis.
Harry Kornfeld, Brooklyn, N. Y.

Hunter McClure, San Francisco, Cal.
Harry Gaillard Riblet, Erie, Pa.
Roger Cushing Rice, Topeka, Kans.
Alston Orange Rose, Pittsburgh, Pa.
William Stava, San Francisco, Cal.
Leon Swartz, East Oakmont, Pa.
Harry Strong Winn, Kansas City, Mo.

The Secretary announced the following deaths:

Daniel Marshall Andrews, of Montgomery, Ala., elected Member, March 2d, 1892; died June, 1917.

Hiram Martin Chittenden, of Seattle, Wash., elected Member, February 7th, 1900; died October 9th, 1917.

Elmer Ellsworth Colby, of Chickasha, Okla., elected Member, January 7th, 1913; died October 4th, 1917.

William Walter Marr, of Springfield, Ill., elected Member, February 2d, 1909; died October 3d, 1917.

Adjourned.

November 7th, 1917.—The meeting was called to order at 8.30 P. M., in the Auditorium of the United Engineering Building; President Pegram in the chair; Chas. Warren Hunt, Secretary; and present, also 120 members and 6 guests.

The minutes of the meetings of September 19th and October 3d, 1917, were approved as printed in *Proceedings* for October, 1917.

A paper by Charles W. Okey, Assoc. M. Am. Soc. C. E., entitled "The Subsidence of Muck and Peat Soils in Southern Louisiana and Florida", was presented by the Secretary, who also read communications on the subject from Messrs. Arthur E. Morgan and Orrin Randolph. The paper was discussed further by Messrs. J. F. Coleman, Rudolph Hering, and T. Kennard Thomson.

A second paper, by H. C. Vensano, M. Am. Soc. C. E., entitled "Pulsations in Pipe Lines, as Shown by Some Recent Tests", was presented by the Secretary, who also read a communication on the subject from Norman R. Gibson, M. Am. Soc. C. E. The paper was discussed, also, by Messrs. Rudolph Hering and R. D. Johnson.

The Secretary announced the following deaths:

William Sinclair Bacot, of Utica, N. Y., elected Member, October 1st, 1890; died October 31st, 1917.

Harry Madera Gould, of Nashville, Tenn., elected Member, November 30th, 1909; died September 30th, 1917.

John Edward Swanker, of Buffalo, N. Y., elected Member, May 4th, 1904; died October 20th, 1917.

Louis Wachtel, of Wells, N. Y., elected Junior, March 2d, 1909; Associate Member, September 2d, 1914; died October 10th, 1917.

Adjourned.

OF THE BOARD OF DIRECTION

(Abstract)

June 11th, 1917.—The Board met at 10.10 A. M.; President Pegram in the chair; Chas. Warren Hunt, Secretary; and present, also, Messrs. Coleman, Craven, Crocker, Davies, Fay, Flinn, Herschel, Hill, Humphreys, Kittredge, Khuen, McDonald, Marx, Noble, Ockerson, Rights, Swain, and Webster.

Mr. Hunt presented a statement signed by himself in answer to the statement presented at the meeting of April 17th, 1917, signed by Messrs. Endicott, Ockerson, and McDonald.

On motion, duly seconded, it was:

"*Resolved:* To expunge from the Minutes all letters and communications in regard to the work of the Committee of the Board on Revision of the Constitution."

The President was requested to appoint a Committee of three to consist of members of the Board whose names have not appeared in the controversy in regard to the Report of this Committee, such Committee to select the matter to be expunged from the Minutes and removed from the files of the Society. The President subsequently appointed Messrs. Alfred D. Flinn, George A. Harwood, and Frederick C. Noble.

The Secretary presented his Financial Report from January 1st to date, together with a certificate of the Auditors.

Reports of Progress were received from the Chairmen of the Finance, Library, and Publication Committees.

The following report was presented by Mr. Davies:

"June 5, 1917.

"THE BOARD OF DIRECTION,
American Society of Civil Engineers,
New York, N. Y.

"Gentlemen: At your last meeting, April 17, 1917, you referred to the Committee on Special Committees two matters for investigation and report, regarding which we beg to submit the following report:

"(1).—*Request for the Appointment of a New Special Committee on Concrete and Reinforced Concrete.*

"We have given careful consideration to the possible advantages to the Society of continuing the Special Committee on Concrete and Reinforced Concrete or of the appointment of a new Special Committee. Since the date of the recent report there is nothing new in the art which would warrant continuing the old, or appointing a new, Special Committee to continue the work beyond the scope of the final report which was presented in January. There is no warrant under our Constitution for maintaining a Special Committee perpetually to watch developments in any particular branch of the art of engineering.

"Our recommendation is that no such Special Committee should be continued nor new Special Committee appointed. In the future,

when there shall have been some further advance in the use of concrete and reinforced concrete sufficient to warrant a further study and report on the subject, the appointment of a Special Committee can be considered on its merits.

"(2).—*Request for the Appointment of a Special Committee of Five Members to Investigate and Report on the Conditions and Opportunities for American Engineers in Russia and in South America.*

"Article VI, Section 12, of the Constitution of the Society, provides that, 'Special committees to report upon engineering subjects shall be authorized, except as further provided in this paragraph, etc.' In our opinion, the matter of investigating the conditions and opportunities for American Engineers in Russia and in South America, does not come within the scope of the meaning of 'engineering subjects', and we, therefore, recommend against the appointment of such a Committee as is requested.

"In submitting this report, we beg to state that while it is not signed by Mr. W. L. Darling, a member of the Committee on Special Committees, the recommendations herein made are fully concurred in by him. The absence of Mr. Darling's signature hereto is due to his leaving this country on government business prior to the report being written.

"Committee on Special Committees,
"J. V. Davies, *Chairman,*
"Geo. W. Tillson."

The recommendations of the Committee were adopted as the action of the Board.

The Secretary presented the Minutes of a meeting of the Executive Committee, held May 8th, 1917, which had already been forwarded to the members of the Board. The action taken by the Committee was as follows:

(1) To abandon the Minneapolis Convention, and to circularize the membership in regard to it.

(2) A resolution to the effect that remittance of dues cannot be made in advance, but that all members of the Society who enter the U. S. Military Service will be carried on the rolls, if they so request, until the war is over.

(3) An appropriation of $1 200 made to the Special Committee on Steel Columns and Struts.

(4) A general report on the sale or lease of the Society property.

The action of the Committee in all these matters was approved.

The Secretary reported in the matter of the panelling of two large rooms in the new Society Headquarters in Thirty-ninth Street that careful estimates had been made for the panelling of the larger of the two rooms in oak and of the Board Room in mahogany, the total cost to be approximately $5 000, and that the work had been ordered.

The action was approved.

The Secretary presented a letter from John C. Hoyt, Secretary, Local Association of Members of the District of Columbia, asking that, in order that the membership may be able to give the fullest consideration to the proposed Revised Constitution, the correspondence ordered spread upon the Minutes of April 17th, 1917, be printed in *Proceedings*, or as a separate publication. The Secretary was directed to inform Mr. Hoyt that the correspondence in question is not now a part of the Minutes of the Board, and to state further that while that correspondence was ordered spread upon the Minutes of the Board of April 17th, it was ordered expunged at this meeting.

Mr. Flinn, Chairman of a committee to report on the proper disposal of the 22 000 duplicates remaining in the Library after its consolidation with that of the U. E. S., reported in favor of disposing of the collection as a whole by presenting it to the Cleveland, Ohio, Association of Members of the Society on the understanding that the collection would be kept intact. This recommendation was adopted as the action of the Board.

President Pegram was authorized to appoint five representatives of this Society on Engineering Council.

The following resolutions were adopted:

"*Resolved:* That the 'General Conference Committee of the National Engineering Societies' now in being, and consisting of Robert Ridgway, S. L. F. Deyo, and Chas. Warren Hunt, is hereby discharged; and

"*Resolved:* That the representatives of this Society in Engineering Council shall, as soon as may be after their appointment, and acceptance of the same, appoint a Secretary whose duty it shall be to report in writing to this Board of Direction, through its Secretary, the Proceedings of said Engineering Council immediately after each meeting of the said Council."

The Secretary reported that he had received many replies from Senators, Representatives, etc., regarding the resolution of this Board on Universal Military Training.

The Secretary reported that the President had named Messrs. J. V. Davies and J. E. Greiner at the request of Dr. Hollis Godfrey of the Advisory Commission, Council of National Defense, and by him had been appointed as members of the Engineering Societies Section of the Committee on Science and Research of the Advisory Commission.

The President was authorized to appoint a Committee to Recommend the Award of Prizes for 1917.

The Secretary presented a card in remembrance of James Forrest, Honorary Secretary, The Institution of Civil Engineers, who died March 2d, 1917, aged 91. Messrs. George W. Kittredge, Clemens Herschel, and J. Vipond Davies were appointed to prepare a suitable response in behalf of this Board.

Director Crocker called attention to the report adopted by this Board in regard to an Employment Bureau, which was referred to the Engineering Council, with the recommendation that it should take up this very important matter.

It was moved, seconded, and carried, that in view of the abandonment of the Annual Convention at St. Paul and Minneapolis, President Pegram be requested to deliver his Address at the time of the Annual Meeting of 1918.

Past-President Swain, as one of the representatives of the Society on the Committee in regard to the adoption of the Metric System, reported, and the following resolution was adopted:

"*Resolved:* That it is the sense of this Board that this Committee should not be enlarged, but should be confined to representatives of the four National Societies."

The question of the use of metric equivalents, as far as practicable, in the publications of the Society, which has been suggested, was referred to the Publication Committee.

The resignations of 1 Associate Member and of 1 Junior were accepted.

Ballots for membership were canvassed, resulting in the election of 6 Members, 40 Associate Members, and 13 Juniors, and the transfer of 7 Juniors to the grade of Associate Member.

Thirty Associate Members were transferred to the grade of Member.

Applications were considered and other routine business transacted.

Adjourned 3.20 P. M.

June 12th, 1917.—The Board met at 3 P. M.; President Pegram in the chair; Chas. Warren Hunt, Secretary; and present, also, Messrs. Coleman, Crocker, Fay, Herschel, Humphreys, Khuen, McDonald, Marx, Noble, Ockerson, Rights, and Swain.

A report from the Membership Committee was received and acted upon.

Adjourned.

September 11th, 1917.—The Board met at 10.30 P. M., immeately after the adjournment of the Membership Committee; President Pegram in the chair; Chas. Warren Hunt, Secretary; and present, also, Messrs. Davies, Fay, and Noble.

Ballots for membership were canvassed, resulting in the election of 17 Members, 63 Associate Members, 1 Associate, and 19 Juniors, and the transfer of 19 Juniors to the grade of Associate Member.

Twenty-one Associate Members were transferred to the grade of Member, and 1 Associate was transferred to the grade of Associate Member.

A Report from the Membership Committee was received and acted upon.

Adjourned.

October 9th, 1917.—The Board met at 10.10 A. M.; President Pegram in the chair; Chas. Warren Hunt, Secretary; and present, also, Messrs. Coleman, Davies, Davis, Duryea, Fay, Harwood, Herschel, Khuen, Noble, Ockerson, Rights, Tillson, and Webster.

Messrs. Crocker, Flinn, and McDonald came in during the session.

The resignation of Director F. G. Jonah on account of his having gone into active service in the Army was received and accepted.

John W. Alvord was unanimously appointed to fill the vacancy for the unexpired term as a representative of District No. 8.

The Report of the Nominating Committee was received.

On motion, duly seconded, a contribution out of the funds of the Society of $500 was made toward the erection of a memorial to the late John Ericsson, Hon. M. Am. Soc. C. E.

A report was received from the Library Committee.*

Mr. Harwood, Chairman of the Publication Committee, reported in the matter which had been referred to it by the Board that the Committee had unanimously decided to recommend that the use of metric equivalents in publications of the Society be not adopted.

The recommendation of the Committee was adopted as the action of the Board.

The following resolution was adopted:

"*Resolved:* That in view of the suggestion of the Committee on Education of the Council of National Defense, that the Engineering Societies through their representation on the General Engineering Committee of the said Council should co-operate with them in connection with the consideration of the subject of Engineering Education: it is recommended to the other National Engineering Societies that this matter, together with the general subject of Engineering Education, be referred to Engineering Council for its active participation and co-operation."

The following resolution with regard to a closer relation between this Society and the Canadian Society of Civil Engineers was adopted:

"*Whereas:* The American Society of Civil Engineers and the Canadian Society of Civil Engineers are both organized for the purpose of promoting, in every practicable way, co-operation, good feeling and fellowship, and the professional and personal interests of the engineering profession in both countries, and for the maintenance of high professional and personal standards and ethics of conduct; and

* See page 671.

"*Whereas:* Many of the members of each of these Societies have become members of the other; and

"*Whereas:* It is in the highest degree desirable that the utmost co-operation and the most cordial relationship should exist between the two Societies, especially in view of the present world crisis, which should draw into closer relationship the United States and Dominion of Canada; be it

"*Resolved:* That the Board of Direction of the American Society of Civil Engineers formally express to the Council and members of the Canadian Society of Civil Engineers, its desire that the two Societies should co-operate for mutual advancement to the greatest extent possible, and that the Board extend to the members of the Canadian Society of Civil Engineers a cordial invitation to avail themselves of the facilities offered by the American Society of Civil Engineers, to make themselves at home in its rooms and to attend its meetings whenever they may visit New York; and further,

"That inasmuch as both Societies are engaged in the effort to solve the same problems, for which purpose mutual acquaintance and intercourse will be of benefit to both Societies, the Board of Direction of the American Society of Civil Engineers expresses its approval of a plan of holding joint meetings of the two Societies at such times and places as may be found convenient; and further,

"That a copy of these Resolutions be sent to the Council of the Canadian Society of Civil Engineers."

Upon the report of a Committee consisting of Messrs. George W. Kittredge, Clemens Herschel, and J. Vipond Davies, appointed to formulate an appropriate entry to be made in the records of the Society on account of the death, on March 2d, 1917, of James Forrest, Honorary Secretary of The Institution of Civil Engineers, the Board adopted the following Minute:

"In view of his unusual connection during seventy-five years with the Institution of Civil Engineers—sixty-one of which were passed as Assistant Secretary, Secretary, and Honorary Secretary—the Board of Direction of the American Society of Civil Engineers respectfully records its appreciation of the good services done by Mr. James Forrest. His useful career was helpful to Engineering Societies and to the Engineering Profession wherever found.

"Another* has said, and, from personal knowledge, it is corroborated:

"'It may truly be said of him that he not only watched over the interests of the Institution with the greatest assiduity and complete success, but that his office was to him a labor of love. Certainly no man ever devoted himself more thoroughly, heart and soul, to his work, than did Mr. Forrest from first to last during his long tenure of office.'

"A brief chronological record is,

1825 Born, November 30th, 1825.
1842 Joined staff of Institution of Civil Engineers.

* Address of John Wolfe Barry, President, Institution of Civil Engineers, *Minutes of Proceedings*, Inst. C. E., Vol. 127, p. 31; *Engineering*, March 9th, 1917.

1845 Awarded a Walker Premium by the Institution for "Drawings and Diagrams Illustrative of Numerous Papers Read at the Meetings". (*Minutes of Proceedings,* Inst. C. E., Vol. 4, p. 4.)
1852 May 4th, balloted for and duly elected as Associate of the Institution. James Meadows Rendel, President. (*Minutes of Proceedings,* Inst. C. E., Vol. 11, p. 477.)
1856 Appointed Assistant Secretary. Robert Stephenson, President. (*Minutes of Proceedings,* Inst. C. E., Vol. 16, p. 187.)
1860 Appointed Secretary. George P. Bidder, President. (*Minutes of Proceedings,* Inst. C. E., Vol. 19, p. 230.)
1896 Retired as Secretary and became Honorary Secretary.
1917 Died, March 2d, 1917."

Mr. Hunt presented a report concerning the finances, furnishing, and building of the new quarters of the Society, which was ordered printed in *Proceedings* for the information of the membership.*

The Secretary reported in regard to the Fifty-seventh Street property that no disposition has yet been made of it. This matter is in the hands of the Executive Committee.

A letter from the American Society of Mechanical Engineers was presented inviting this Society to appoint three of its members to join with similar representation from the Prime Movers Committee of the National Electric Light Association, the American Institute of Electrical Engineers, and the Committee on Power Tests of the Am. Soc. M. E., to assist in framing a Code for the Testing of Water Wheels. The President was empowered to appoint such Committee.

The appointment as representatives of this Society on the Engineering Council of Messrs. John F. Stevens, George F. Swain, Frederick H. Newell, Alexander C. Humphreys and John D. Galloway, was reported.

Director Flinn stated that he is a member of a Committee on Personnel appointed by Engineering Council, and that for this committee a partial list of "Civil Engineer Specialists for War" has been compiled, and that he has been requested to ascertain whether the Board of Direction would provide for and authorize the sending to all members of the Society a questionnaire on this subject.

The Board directed that there be published in *Proceedings* a statement of the work of the Committee of which Mr. Flinn is a member, and that all members of the Society who are available and desirous of serving the Government in the present crisis be asked to classify themselves under the various heads, so that their names may be listed by the Committee of the Engineering Council with a view of aiding the Government.†

* See page 666.
† See page 664.

The appointment of Messrs. J. L. Van Ornum, H. J. Burt, and S. E. Tinkham as a Committee to Recommend the Award of Medals and Prizes for 1917 was reported.

The Secretary stated that the following cable had been received from John F. Stevens, M. Am. Soc. C. E., dated July 31st, 1917:

"Central Board Engineers Ways of Communication send Fraternal Greetings to you request me to thank United States through Society for sending our Advisory Commission to Russia."

Also, that President Pegram had transmitted this to the President of the United States, July 31st, 1917, and had received a reply stating: "The President has asked me to thank you warmly for your kind letter of July 31st. He is very much gratified by your generous words."

A letter from E. P. Goodrich, M. Am. Soc. C. E., Director of Military Census, New York City, was presented, expressing appreciation of himself and colleagues for assistance given by the Society in the work of taking the State Military Census and Inventory. The Secretary also reported that he received a similar letter from Mayor Mitchel.

The Board authorized the appointment of a Committee of the Board to canvass the Preliminary Suggestions for Members of the Nominating Committee on November 1st, 1917, and also fixed the date of December 14th, 1917, for mailing the form for Final Suggestions for Members of the Nominating Committee, and January 15th, 1918, as the date for the final canvass of those suggestions.

The date of the next Board meeting was fixed as January 14th and 15th, 1918.

George H. Pegram was appointed as one of the representatives of this Society on the John Fritz Medal Board of Award to fill the vacancy caused by the retirement from that Board of George F. Swain, whose term expires January 18th, 1918.

Mr. Hunt presented a Report from the Alfred Noble Memorial Committee, which was received and ordered printed in *Proceedings* for the information of the membership.*

The Secretary reported that it had been impossible to secure a list in any way complete of the members of the Society who are in the service in the Army and Navy, and recommended that he be authorized to issue a circular addressed to all members asking for special information in this matter in order that a "Roll of Honor" may be published in *Proceedings*. The authority requested by the Secretary was granted, and it was further decided that such list should be kept up to date and published regularly in *Proceedings*. It

* See page 648.

was further decided that this Roll of Honor should be restricted to those holding commissions or official appointments.

A Committee consisting of Alfred D. Flinn, George A. Harwood, and Frederick C. Noble, appointed by the Board at its meeting of June 11th to select matters to be expunged from the Minutes of the Board and removed from the files of the Society in regard to a controversy in connection with the Report of the Committee on Revision of the Constitution, reported that it had unanimously directed the Secretary to expunge from the official Minutes of the Board of Direction the following matter:

1. A minority report dated February 1st, 1917, signed by three members.

2. A majority report dated March 1st, 1917, signed by four members.

3. Letter dated March 19th, 1917, from Dr. Hunt to Mr. Pegram.

4. Letter dated March 22d, 1917, from President Pegram to Dr. Hunt.

5. A document, dated April 17th, 1917, handed to the Secretary on that date and read to the Board, signed by M. T. Endicott, J. A. Ockerson and Hunter McDonald.

6. Statement by H. S. Crocker made verbally at the meeting of the Board April 17th.

7. Letter dated March 23d, 1917, from the Secretary to members of the Board of Direction transmitting proofs of the proposed Constitution and copies of Items 3 and 4 in the above list;

And to remove from the files of the Society the seven items just enumerated, together with:

8. A written communication dated June 4th, 1917, signed by Dr. Hunt, in answer to the statement made by M. T. Endicott, J. A. Ockerson and Hunter McDonald, which was presented to the Board June 11th, 1917, but not made a part of the official minutes;

And to forward all of the above mentioned documents to the Chairman of this Committee, Alfred D. Flinn, who shall hold said documents until the Committee decides when they shall be destroyed.

It was moved and seconded that the Report of the Committee be received and its recommendations adopted.

A roll-call on this motion was called for.

The following members voted to adopt the recommendations of the Committee: Messrs. Crocker, Davies, Duryea, Flinn, Harwood, Khuen, Noble, Pegram, Rights, Tillson, and Webster—11.

The following voted not to sustain the Committee: Messrs. Coleman, Davis, Fay, Herschel, McDonald, and Ockerson—6.

Mr. Hunt did not vote.

The President declared the motion adopted.

The Secretary presented a letter from Onward Bates, Past-President, in regard to the proposed revised Constitution. He also stated that he had letters from Messrs. Leonard Metcalf, and W. S. Richmond, suggesting modifications.

It was moved that Mr. Bates' letter be published in *Proceedings*, and also that all communications in regard to the revised Constitution be published in *Proceedings*. These motions were not voted upon, but after much discussion the Secretary was instructed to advise Mr. Bates and those who had sent in comments that their letters had received careful consideration, and that the matters in them have been referred to the Publication Committee for consideration, and for proper presentation to the entire membership of the Society.

Mr. McDonald presented the following resolution:

"*Resolved:* That at least 25 days prior to the Annual Meeting a notice be sent out to all Corporate Members giving the proposed revision of the Constitution, signed by certain members of the Board, together with a statement of the proposed changes and the reasons therefor, and that the Committee on Publication be authorized to prepare this statement."

The following members of the Board then authorized their names to be signed to the revised Constitution: Messrs. Ockerson, Herschel, Fay, Coleman, Webster, McDonald, Tillson, Davies, Harwood, Khuen, and Davis—11.

The other members of the Board present declined to allow their names to be so used.

The resignations of 4 Associate Members were accepted.

Ballots for Membership were canvassed, resulting in the election of 7 Members, 61 Associate Members, and 17 Juniors, and the transfer of 19 Juniors to the grade of Associate Members.

Thirteen Associate Members were transferred to the grade of Member, and 1 Associate was transferred to the grade of Associate Member.

Applications were considered and other routine business transacted.

Adjourned.

October 10th, 1917.—The Board met at 11.07 A. M.; President Pegram in the chair; Chas. Warren Hunt, Secretary; and present, also, Messrs. Coleman, Crocker, Duryea, Fay, Flinn, Humphreys, Khuen, and McDonald.

A report from the Membership Committee was received and acted upon.

Adjourned.

SOCIETY ITEMS OF INTEREST

Alfred Noble Memorial

To the Board of Direction,
American Society of Civil Engineers,
220 W. 57th Street,
New York City.

Gentlemen: Your Committee on the Alfred Noble Memorial desires to make a progress report, and believing that the Members of the Society are entitled to some account of our work during the three years of this Committee's existence we have recounted the history of it in the form which follows, with the request that if it meets the approval of your Board, you will direct it be sent out to Members for their information.

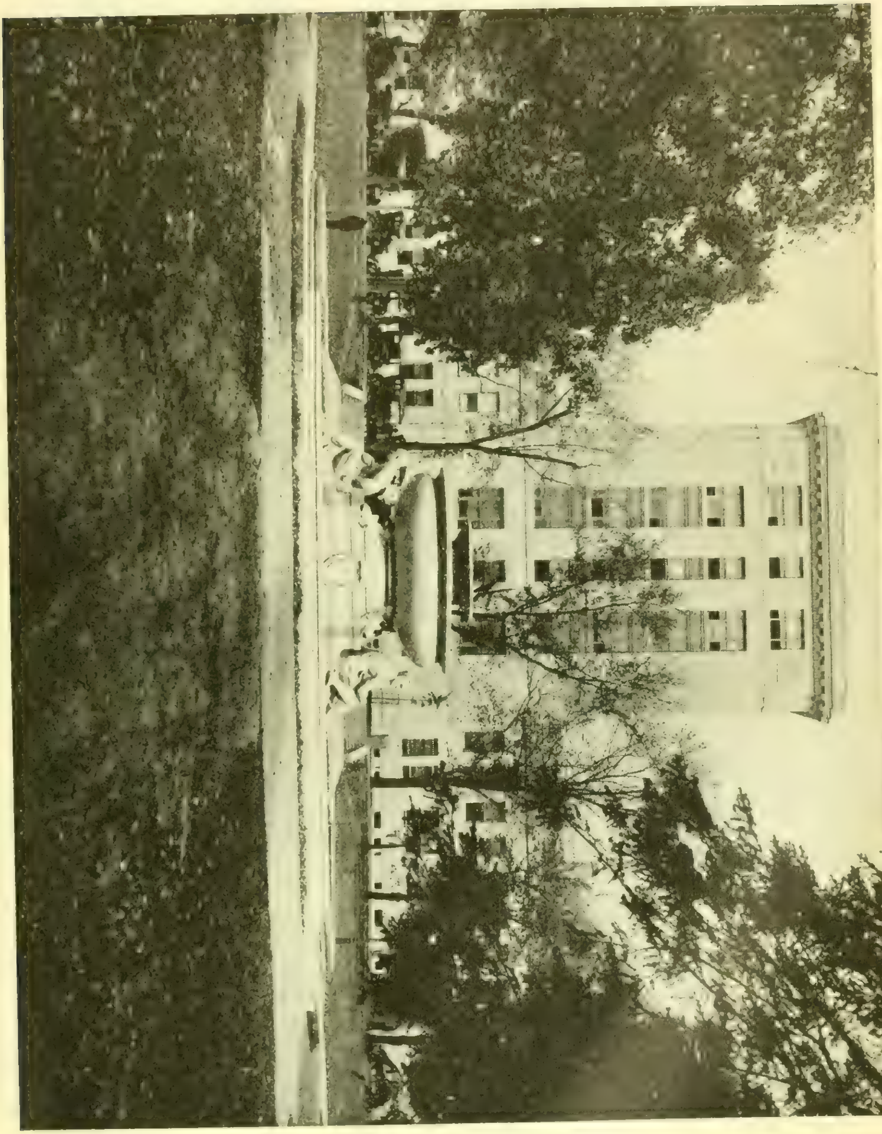
During a meeting of the Board of Direction of the American Society of Civil Engineers held on June 2d, 1914, a tribute by the engineering profession to the life and memory of Alfred Noble was unanimously adopted and spread upon the minutes. The Board at the same time thought it proper and desirable to recognize the worth of the man in a more substantial way by erecting a memorial monument to him. This was first brought to the attention of the members of the American Society of Civil Engineers by the circular of the Secretary dated July 1st, 1914.

Memorial a National Honor.—It was decided at that time, that as Alfred Noble's professional practice was national in character and as he was an adviser of the President and other officials of the Government and was consulted on some of the most important engineering work of the country, the Capital of the Nation was the proper location for such a memorial. The Board set aside the sum of $1 000 as a first subscription towards the necessary funds, and, to get work started, appointed the following committee with power to carry out the project:

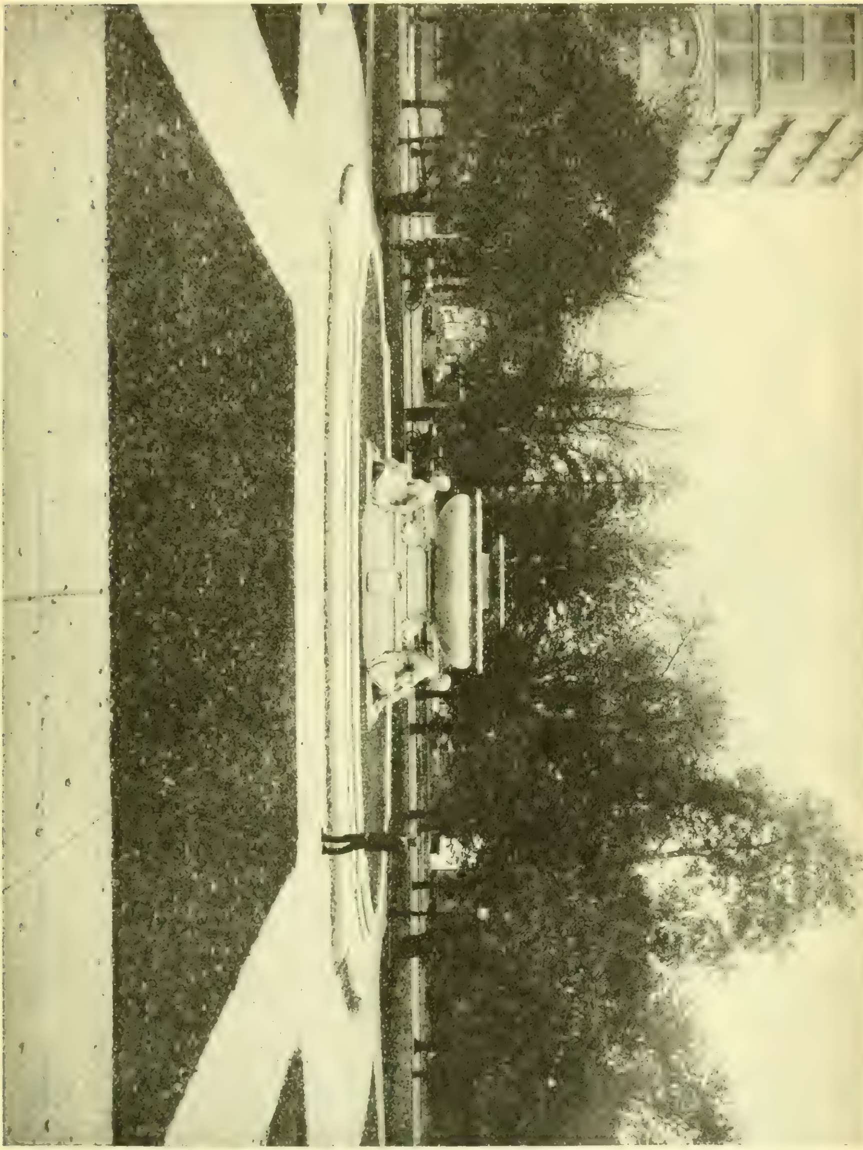
Committee Appointed

Messrs. Onward Bates, Chicago, Illinois, Chairman
Robert Moore, St. Louis, Missouri
Samuel Rea, Philadelphia, Pennsylvania
Samuel H. Hedges, Seattle, Washington
F. H. Newell, Washington, D. C.
W. W. Harts, Washington, D. C.
Chas. Warren Hunt, New York City, Secretary

On December 6th, 1916, Mr. Robert Moore resigned from the Committee stating that it had been impossible for him to attend any meetings of the Committee, or give to the subjects committed to them



View to the North. Interior Department Building in Background.



VIEW TO SOUTH.

such attention as they certainly deserved, nor would it be possible for him in the future to do any better, and the Board felt bound to respect his wishes. Hugh L. Cooper, M. Am. Soc. C. E., of New York City, was appointed to serve in his stead.

Architectural Adviser.—Following the customary proper procedure in such cases, as recommended by the National Commission of Fine Arts, the first step of the Committee was to employ a competent architectural adviser to advise them as to the best site for the purpose in the City of Washington and the best memorial treatment of that site. The architect selected was Mr. Glenn Brown of Washington, who had been recommended as a man of eminence in his profession and through his intimate touch with the established plans for the harmonious development of the National Capital well fitted for this important duty. The following report of the architectural adviser explains the steps taken:

Report of Architectural Adviser.—"In connection with the location of the Noble memorial I have studied many of the triangular parks, like those on Pennsylvania Avenue, the park south of the War Department, the advisability of planting it somewhere on the Mall, and the approaches to the important bridges.

"After careful consideration I have determined that such a memorial could not appropriately be placed on any of the above mentioned sites,—in some cases because of the area being insufficient, in others because I have felt no monument should be placed in the park, their form not being suitable for such adornment,—and in others because of a lack of appropriateness in placing an engineer in an isolated position with unsympathetic surroundings.

"For many years I have felt that small statues and monuments in Washington have been placed in a haphazard and thoughtless way about the city. All I think will agree that some plan should be devised by which they could be grouped, not as monotonous and detrimental adjuncts, but so that they would increase the interest and add to the beauty of buildings and landscape. We might group literary men around the Library, statesmen around the Capitol, warriors at the War College, scientific men in connection with the Smithsonian Institute, etc.

"While a plan of this kind might easily become disastrous, yet if competently designed as a part of the park in scale with the buildings there can be no question but that it would add to the interest and beauty of this city.

"This idea has been one of the controlling factors in my selection of Rawlins Park adjoining the Department of the Interior on the south, as the proper site for a memorial to an eminent engineer selected by his associates as one fitting to be commemorated in the Capital City, as this Department contains several of the important engineering

branches of the Government. I feel that important streets interrupted by the park and the Interior Department Building, the most imposing structure in the vicinity, should be recognized in the placing of this statue, and in the planting and the walks of the park. It will be acknowledged by those who have studied the question that the parks and their adornments have been treated in most cases as distinct entities, separated from the city and not a part of the city system. As I consider this method wrong for parks in the city intimately connected with the streets and buildings, I am recommending that Rawlins Square have its walks and planting and the memorial located so as to relate to such streets as New York Avenue and the new Interior Department Building, thus making it a harmonious part of its surroundings. While my first thought was to place the memorial on the south of the square on the axis of the Department, with an open view bounded by trees and some formal planting on lines following the central part of the building, this required a walk deflecting the line of New York Avenue which will, as the Riverside Park is completed, become one of the important drives of the city. I have recommended the carrying of the line of New York Avenue through the park, not as a street, but as an open view, with trees planted far enough apart to prevent the obstruction of this view in the future and with the grass, plants and walks on this axis while connecting the park with the system of the city streets also gives the most direct line for foot traffic.

"To connect the design with the Departmental Building I propose that the center of the park leading up to the memorial be confined to low rather formal planting following the lines of the center portion of the building, thus giving an open view by their future growth. I find by an examination of the trees in this park that the largest part are maples in the last stages of decay—lacking in form and beauty; others, while sound, are not of sufficient importance to prevent a replanting of the park to join it to its surroundings.

"As to the character of the memorial I recommend that it take the form of a fountain with large scale architectural features and forceful symbolic figures, typifying the joining of the Great Lakes, one of Alfred Noble's greatest achievements in the public service. I would also recommend that the representation of the engineer be confined to a low relief which will form a part of the composition. This will give the sculptor an opportunity to express his ideas in an ideal and poetic way and not confine him to the prosaic use of modern costume.

"I submit with this sketches in explanation of my recommendations of the relation of the park to the memorial and the suggested character of the memorial, not with the idea of binding the artist to a fixed design but simply to explain my report clearly.

"I have determined to recommend Mr. Paul Bartlett as the sculptor to do the work and would recommend leaving to him the selection of a design for architectural features.

"I recommend Mr. Bartlett because, while an American, he has an established reputation abroad and now has one of the most notable pieces of sculpture to be erected in this country in the pediment of

View Looking Directly East.

View Looking Directly West.

the Capitol, and full size models showing skill, taste, and good judgment.

"This work has caused him to live in Washington a large part of the past few years and has brought him in sympathy with the development of the city."

Site Adopted by Committee.—Upon receipt of this report the Committee held a meeting in Washington, going over the report and inspecting the site recommended by the adviser. The site appeared to them a particularly fortunate one, and was accepted without delay. Before formal adoption of these recommendations by the Committee the National Commission of Fine Arts had as a matter of courtesy reviewed the recommendations of the expert adviser as to location and general scheme, and found them acceptable as promising the best artistic results.

Sculptor Selected.—The question of selecting a satisfactory sculptor was then taken up. The expert adviser had recommended as his first choice Mr. Paul Bartlett but had also submitted two alternative names. Before accepting the adviser's recommendation the Committee inspected photographs of the executed works of many other sculptors than the three mentioned and later inspected some of Mr. Bartlett's executed works. The Commission of Fine Arts, in acceding to the request for an opinion as to the site, the type of memorial suggested, and the choice of a designer, had approved all as recommended by our architectural adviser; and in reviewing the sculptors suggested by him had stated that any selection from the list prepared would give the Society good results. Mr. Bartlett was selected and the offer of the commission made to him. He accepted, and chose as his architectural collaborators Messrs. Glenn Brown and Bedford Brown, the first of whom had acted as expert adviser to the Committee.

Design Proposed.—The sculptor and architects proceeded immediately upon their study of the problem and after preparation of many plans and studies and models, not only of the fountain itself but of the whole part treatment, finally evolved the design which is submitted herewith. They describe the result of their efforts as follows:

"The designs shown in the models for the Alfred Noble Memorial, while they commemorate the character and accomplishments of the great engineer, are regulated by the act of Congress fixing its character as a fountain and by its location in Rawlins Square, south of the new Interior Department Building.

"The center of the square, after thorough study, was considered the best location. The planting suggested for the memorial shows an open court facing the central portion of the new Interior Department Building, giving the most imposing view from this point, and binding it in with this Department in which the principal engineering

branches of the Government are housed. The diagonal lines direct for traffic are kept open, giving a diagonal view of the memorial, the second most imposing view. The oblong character of the fountain was adopted because it seemed to fit most logically with the form of the Square. It was determined that it should be made low and massive as befitting the solidity and permanence of engineering work. After studying and discarding many compositions based on the above ideas, the models shown here have been selected as combining best the features desired—a fountain befitting the site, and a memorial to an eminent engineer, symbolizing by its form and sculpture the character of the man and his life.

"The dominant architectural element is the large elliptical bowl, set on a solid base and flanked by the dominant sculptural element, two strong seated figures. The memorial will rise from a large basin of water, 37 ft. wide and 63 ft. long, surrounded by a wide stone coping, indicating the form of the composition, making it a part of the memorial, and for this reason dominating the water surface.

"The sculptural figures are intended to show the strong elements in Alfred Noble's character—thought in preparation, force in execution. Each figure will hold a scroll upon which will be carved an example of the work by which Alfred Noble is so well known and appreciated. Properly placed on the coping beneath the figures will be inserted descriptive lettering. Directly below the name on the base will be a large cartouche, on which there will be a relief portrait standing out prominently in the central position on each of the long sides of the memorial.

"The display as a fountain will be imposing and attractive. The water will rise in a large mass, falling into and overflowing from a small bowl into the large oval bowl. From the large bowl it will overflow into the great basin at the ground level. Water jets will flow into the large basin from the dolphins on the base. Minor jets are proposed on the long axis in the subordinate centers of the large basin. While this play of water will make the memorial attractive as a fountain, the sculptural and architectural composition will be an imposing memorial independent of the flow of water.

"To give the memorial distinction and avoid the combination of bronze and stone, which has become commonplace, it has been determined to use stone for both the architectural and sculptural features, so we can easily obtain harmony in color between the sculpture and architecture and emphasize and contrast important features of the design by different methods of tooling. Granite is the most desirable because of durability and the easy possibilities of contrasting methods of surface treatment. The color which is considered preferable is a green granite, like the Rockport granite. It is proposed to have the architectural portions fine cut rubbed or honed, while the sculptural work will be honed, combined with polishing high lights.

"As a statue in modern clothes looks crude and in a little while ridiculous, a realistic treatment has been avoided and the memorial will perpetuate the work and express the character of Alfred Noble, as by his mind and its products future generations will know and honor him.

View Looking Northeast on the Axis of New York Avenue.

View Looking Southwest on the Axis of New York Avenue.

View Looking Northwest on the Axis of Virginia Avenue.

View Looking Southeast on the Axis of Virginia Avenue.

"To make the memorial individual and intimate a portrait relief, showing his physical features and expressing his inner spirit has been given the most prominent place in the composition."

Cost.—The Memorial as planned will cost $45 000. This is for the main memorial composition itself. The United States Government at its own expense has already undertaken the development of the park, which will permit the proper setting for the fountain when erected in accordance with the artist's plans.

Approved by the Committee and the Fine Arts Commission.—It is believed that the design when executed will be dignified and beautiful, and an object of pride to the profession. It has met with the approval of the National Fine Arts Commission, under whose jurisdiction the decision as to the artistic character of memorials in the City of Washington has been placed by law, which also ensures a work of art of the first order.

Location Authorized by Law.—Through the efforts of your Committee, Congress, on May 8th, 1916, granted the following permission to the American Society of Civil Engineers:

"Resolved by the Senate and House of Representatives of the United States of America in Congress assembled, That the Chief of Engineers, United States Army, be, and he is hereby authorized and directed to grant permission to the American Society of Civil Engineers for the erection on public grounds of the United States in the City of Washington, District of Columbia, other than those of the Capitol, the Library of Congress, and the White House, of a memorial fountain to Alfred Noble, a civil engineer of distinguished ability in connection with Government work, whose services have been of conspicuous benefit to the country: Provided, That the site chosen and the design of the memorial fountain shall be approved by the Commission of Fine Arts, and that the United States shall be put to no expense in or by the erection of the said memorial fountain: Provided further, That if the erection of this memorial fountain shall not be begun within three years from and after the passage of this resolution the permission granted may, in the discretion of the Chief of Engineers, be revoked at any time."

Every provision except the last one of this legislative enactment has been complied with. If work is not undertaken within three years the permission to erect this memorial may be withdrawn, and the site be made available for the erection of some other memorial for which there is always great pressure.

Present Condition of Work.—This is shown by the illustrations accompanying this report, which are from photographs of a full size

model of the Memorial, made in plaster. The United States Government has already graded the park and built permanent cement walks as planned by the architect, and the planting of trees and shrubbery will follow at the proper time. The Memorial in plaster will last until it can be replaced by permanent granite. In the meantime, engineers, and the public generally, have the opportunity of judging the quality and appropriateness of the design, which will be closely followed in granite, subject to minor changes which may be proposed by the sculptor or the National Fine Arts Commission.

Time.—The time required for the construction of a work of art is disappointing to engineers, accustomed to regard time as an essential factor in construction. Your Committee has sometimes chafed under what appeared to be unnecessary delays, but it believes we have moved as speedily as in the case of other monuments in the City of Washington, and having reached a satisfactory conclusion as to the size and character of the Memorial, it feels that, on looking backward, it has accomplished as much progress as could be expected under the conditions which have prevailed in the three years since it was appointed. If the conditions were favorable we would now be asking for subscriptions and proceeding with the permanent work. All Members know without explanation from the Committee that the present is not a good time to invite engineers to subscribe, nor to attempt the quarrying, the transportation and the carving and erection of the granite work. When the right time comes, and we devoutly hope it is not far away, we will be ready to proceed with all reasonable progress. At present we ask engineers to keep in mind the wish to finish this Memorial designed to honor the profession as it was represented by Alfred Noble. Your Committee is ready at the return of normal conditions to complete its task in a manner that we believe will be gratifying to the profession.

Respectfully submitted,

Onward Bates
Samuel Rea
Samuel H. Hedges
F. H. Newell
W. W. Harts
Chas. Warren Hunt

The Committee has been unable to secure the signature of Maj. Hugh L. Cooper, who is in the Government service "somewhere in France."—*Secretary.*

Civil Engineers for War Service

The United States Government is making demands constantly for civil engineers for service in the United States and abroad, and though hundreds are now in the service, as commissioned officers or civilians, more are needed, and will still be needed after the war ends, to carry on reconstruction and improvement operations on a large scale, in the countries where the armed conflicts have been waged.

One of the functions of the Engineering Council, recently organized by the four Founder Societies, is to collect and classify information about engineers in all branches of the profession, and to aid the Government in securing the right men as needed. This information is being assembled, and is to be kept in a central place, probably the Engineering Societies Building in New York City.

The American Engineering Service Committee, appointed by the Engineering Council for the purpose of obtaining information about engineers, has already been enabled to aid the Government in many cases.

Some of the engineering societies have been collecting information by sending question sheets to their members. So many of such question sheets have been sent out by various organizations that they have become a burden to not a few engineers. Therefore, at a recent meeting of the Board of Direction, it was voted that, instead of sending a question form to all members of this Society, this notice should be printed in the *Proceedings,* inviting all those interested to send to the Secretary specific information concerning their willingness to accept Government service at home or abroad, their preferred specialties, their fitness and experience therein, and approximately the rate of remuneration that would be accepted. For convenience of use, all statements received in response to this notice must be classified and indexed. It will expedite this work if those who respond to this notice will use, as far as practicable, the designations of the brief classification printed below; if these are wholly inappropriate in certain unusual cases, other designations, of course, may be used.

Pending the formation of a list of civil engineers available for war service from the responses to this notice, the representative of the American Society of Civil Engineers on the American Engineering Service Committee, with the assistance of a number of members residing at focal points of engineering activity, has prepared a partial list of "Civil Engineer Specialists for War" who might be available for various kinds of service, especially men of experience who could give advice in different problems or suggest names of men suited to subordinate positions. An important object of this notice is to inform

the membership at large of the existence of this partial list, and the desire to create a more complete list, so that all who wish to be considered for Government service may have their names on file in a place to which inquiries for engineers will be directed. Of course, it cannot be guaranteed that such filing of one's name will secure a Government engagement, or even that it will surely bring some coveted opportunity. It may help both the individual and the Government.

Classification
To be Used in Connection with
Listing Civil Engineers for War Service

NOTE.—In replying, members may use the letters and figures in designating the special work for which they are qualified.

A.—Construction Work, Contracting
1 Earthwork
2 Quarrying
3 Rock excavation
4 Highways
5 Railroads
6 Steel bridges
7 Steel frame buildings
8 Reinforced concrete buildings and bridges
9 Concrete and stone masonry
10 Water-works
11 Sewerage

B.—Structural Engineering
1 Concrete and stone masonry
2 Reinforced concrete
3 Timber construction
4 Arches:—concrete, stone, brick, steel, iron
5 Trusses, girders, building frames, roofs
6 Movable bridges
7 Cantilever and suspension bridges, viaducts
8 Bridge-shop practice, shop inspection
9 Bridge erection, falsework, equipment
10 Bridge painting
11 Coal storage plants
12 Grain elevators
13 Factories and mill buildings
14 Fire prevention and protection

C.—Foundations
1 Underpinning, shoring
2 Timber piles
3 Concrete piles
4 Metal piles
5 Pneumatic and open caissons

D.—Surveying
1 Topographic surveying
2 Geodetic surveying, triangulation
3 Diamond and shot drilling, wash-boring

E.—Railroads
1 Location
2 Construction, and maintenance of permanent way
3 Yards and terminals
4 Signals and signaling
5 Traffic management
6 Mountain railways
7 Aerial tramways
8 Industrial railways

F.—Street Railways
1 Location
2 Construction of track, maintenance
3 Electrolysis, leakage

G.—Municipal Planning
1 City and town planning
2 Industrial towns

H.—Highways
1 Location
2 Earth and sand-clay roads
3 Water-bound surfacing
4 Bituminous pavements and surfaces
5 Cement concrete pavements
6 Brick and block pavements (stone, wood, asphalt)
7 Maintenance

I.—Hydrology, Hydraulics, Dams
1 Hydrology, water resources
2 Earthen, hydraulic-fill, rock-fill dams
3 Timber dams
4 Masonry, reinforced concrete dams
5 Movable dams

J.—Waterways
1 Dredges and dredging
2 Coast erosion and protection
3 Breakwaters and jetties
4 Harbors, roadsteads, and anchorages
5 Bulkheads, dock and quay walls, docks and piers, wharves
6 Canals
7 River regulation and canalization

K.—Water-works
1 Purification of water
2 Aqueducts, conduits, open channels
3 Pumping plants
4 Stand-pipes and tanks
5 Mains and services
6 Rural and isolated water supply

L.—Water Power
1 Water power, hydro-electric plants

M.—Sanitation
1 Sewerage systems
2 Sewage disposal
3 Camp sanitation
4 Refuse disposal
5 Industrial and factory sanitation

N.—Tunnels and Shafts
1 Shaft sinking
2 Tunneling and drifting
3 Timbering, metal supports
4 Masonry lining, grouting
5 Drainage, unwatering

Report of the Secretary on the Society's New Headquarters

October 5th, 1917.

To the Board of Direction,

American Society of Civil Engineers:

The undersigned respectfully reports on various matters which, as Secretary of the Society, and as a member of the Building Committee of the U. E. S., have been under his care since building operations were undertaken last year.

The three stories added to the United Engineering Building had to be carried on steel columns running through the building, inasmuch as the walls were not built to carry additional load. This necessitated a preliminary contract for this work. The Wells Construction Company carried this work out on a percentage basis, the approximate cost being $60 000.

Due to various causes, it was not possible to get plans and specifications ready so that the second contract could be put out for competitive bidding, and the prices of all material were advancing so rapidly that an agreement was entered into with the Wells Construction Company, after careful figuring, on the basis of the then prices (November, 1916) at an upset figure including his percentage of 7%. We were fortunate in being able to secure the steel with only about 30 days' delay in delivery.

The original estimate of the cost of the addition at the time the matter was first proposed was about $210 000, but, in order to cover contingencies, $225 000 was considered ample to carry out the work. The American Society of Civil Engineers agreed to pay the cost of the building, not to exceed $250 000.

The building will cost $300 000, the additional $50 000 being met by assessments on the four Founder Societies, so that the total amount to be paid by this Society will be $262 500.

All of this has been paid with the exception of the additional assessment of $12 500.

It is a matter of congratulation that this work has been carried to completion without any serious accident of any kind, and without very serious inconvenience to the many societies occupying the building.

The lay-out of the floors to be occupied by this Society was made by the undersigned with a view to utilizing every available foot of space and to secure good light. This was the more necessary inasmuch as the area of these two floors is much less than that of the lower floors.

Briefly, the Society will occupy the entire 15th floor, and about two-thirds of the 16th or top floor. In all there are 11 main rooms. On the 15th floor there are:

(1) The office of the Secretary, entrance to which is at the right of the elevators.

(2) The Reading Room, directly opposite the elevator, the entrance to which will be the main entrance to the Society rooms. This room is 51 by 26 ft., and looks out over Bryant Park to the north. It is paneled in oak, and when used by our members, in connection with the Library, will, it is believed, practically take the place of the present Reading Room in 57th Street.

(3) The Board Room. This room, which is 43 by 24 ft., is on the south side of the building, directly opposite the Reading Room, a 6-ft. hallway separating them. This room is paneled with mahogany, and the furniture for it, which has been specially designed, is also of mahogany, and consists of 4 tables and 30 chairs. The tables are designed so that they can be placed together making a table 24 by 6 ft., or can be separated and used as units 6 by 6 ft., and when necessary can be made into tables 6 by 3 ft. to set against the wall and take up very little room. In the partitions between these rooms and the hallway, two 8-ft. openings opposite each other, with sliding doors, have been arranged, so that the two rooms can be thrown together, practically forming one large room averaging 57 by 47 ft.

(4) General Office. A large room covering the east side of the building 59 by 37 ft. Here will be located the general office force. A service stairway, which will practically be a private stairs for this Society, gives access to the 16th floor, where, on the east side of the Building, there are four small offices, as follows: (5) Rest Room for women, (6) For Bookkeeper, (7) Editorial Department, (8) Applications Department.

Three other large rooms are available for Committee rooms, or whatever use may develop in the future. They are: (9) 24 by 20 ft., (10) 22 by 24 ft., (11) 36 by 23 ft., these figures being approximate.

A doorway in the hall separates that part of the floor to be used by the Society from 3 rooms which are available for renting by the U. E. S., and to which access is obtained through the elevator and hallway without passing through the quarters of the Society.

In the budget of last year an item of $20 000 was placed to cover the necessary furniture for the new quarters, and subsequently the undersigned was authorized to contract for the paneling of the Reading Room and Board Room, the estimated cost of which was $5 000, not including architect's fee, the total extra appropriation being $25 000.

It has been found that a great deal of the furniture, which has been in use for just 20 years, is perfectly good, if renovated; in fact, it is doubtful if it would be possible to secure such excellent material at the present time, and it is certain that the expense would be very great. All the furniture in our present Reading Room has been entirely done over, and will be used in the new Reading Room.

In 1897 we had 8 rugs made; these have been in constant use for 20 years, and they are still in very good condition. To give some idea of the changes in price, a large rug which was made specially for use in our Reading Room, and for which the Society paid $450, has been stated by one rug man to be worth about $2 000.

A few extra desks and chairs have been purchased to replace some which were not fit to renovate. The total approximate amount which will be spent for paneling and furniture, including metal shelving, is $13 222, or a little more than half the appropriation.

Attached to this report are:

(A) A statement of the financial condition of the Society on October 1st, and of the obligations which it will have to meet to December 1st, which shows a total estimated deficit of $26 086.71 on December 1st, 1917.

(B) Statement of the cost of furniture.

The actual building operations would have been very nearly finished about the first of October, but the paneling in the two large rooms, which is not part of the original contract, was undertaken as special work for this Society, which would necessarily delay our moving in for some time. In view of these circumstances, the Secretary agreed with the President of the U. E. S. that all interest charges on amounts advanced by the Society (amounting to about $4 891) should cease on October 1st, and that the Society should begin the payment of rental on November 1st. This practically gave us a month in which to get settled in our new quarters. I hope the members of the Board will be able to visit the new building during the day.

Inasmuch as a final draft of $15 000 on the mortgage will carry the Society through very well for its ordinary expenses and the payments of all bills for furniture and moving, until after the first of December, when it will be in funds, I took up the question of the date of payment of the $12 500 still due the U. E. S. with Mr. Chas. F. Rand, President, U. E. S., and he stated to me that it would be satisfactory if this payment were deferred to December at some date prior to the 10th, and I recommend that this action be taken, because if we do not draw the additional $12 500, it can then easily be spared out of current funds, and our total mortgage will be only $150 000, and, inasmuch as the total cost of building, extra work, furniture and moving, will bring the total amount to be expended by the Society up to approximately $276 000, $126 000 of it will have been paid out of current funds.

Respectfully submitted,

CHAS. WARREN HUNT,
Secretary.

(Statements (A) and (B) follow on the next page.)

(A)

STATEMENT RE FINANCIAL CONDITIONS
OF AMERICAN SOCIETY OF CIVIL ENGINEERS
Oct. 1st to Dec. 1st, 1917.

Paid to United Eng. Society (including interest of $4 891.67)		$250 891.67
United Engineering assessment		250 000.00
Amount due Am. Soc. of C. E. from U. E. S. Oct. 1st, 1917.		$891.67
Balance in bank, Oct. 1st, 1917	$6 594.97	
Receipts Oct. 1st, 1917, to Dec. 1st, 1917 (on basis of 1916)	9 210.00	
	$15 804.97	
Ordinary disbursements, Oct. 1st to Dec. 1st, 1917 (on basis of 1916)	17 061.25	
Deficit (on account of current expenditures)	$1 256.28	
To be provided for:		
Furniture, rugs, etc. (as per attached list)	$7 222.10	
Paneling	5 500.00	
Moving	500.00	
One-quarter assessment U. E. S. ($50 000)	12 500.00	$26 978.38
Total deficit Dec. 1st, 1917 (estimated at)		$26 086.71
(Cash receipts Dec. 1st to Dec. 10th, 1916)	$13 548.00	

(B)

FURNITURE ACCOUNT.

Linoleum (estimate)		$554.00
Mahogany furniture, Board Room		2 800.00
Rug for Board Room		832.00
Renovating old furniture (estimate)		563.00
Drawing Board		25.00
Furniture from Quick and McKenna		672.55
Furniture from Stanton M. Child:		
Stationery cabinet	$56.00	
Lockers	160.00	216.00
Metal shelving		700.00
Cleaning of rugs		103.00
Furniture for Rest Room (estimate)		100.00
		$6 565.55
Add 10% for contingencies		656.55
		$7 222.10

Report of the Library Committee, American Society of Civil Engineers.

"October 2, 1917.

"To the Board of Direction,
American Society of Civil Engineers,

"The Library Committee's duties and responsibilities have been much curtailed by various actions of the Board and by events with which the Board is familiar, as compared with the requirements of report is due from it at this time."

"The books and pamphlets of the Society's library which were not duplicates of those already in the United Engineering Library have been moved to the latter library and shelved temporarily, pending the completion of the changes in the library space in the Engineering Societies' Building. The work of comparing the libraries to sort out the duplicates, and the moving of the 67 000 accessions from the Society House to the U. E. S. Library and placing them on the shelves ready for continuous use, while paid for by the U. E. S., was carried out under the direction of the Secretary of this Society.

"Of the duplicates, 22 000 have been disposed of, as directed by the Board, by delivering them to the Cleveland Association of Members of the American Society of Civil Engineers. They have been shipped to Cleveland and placed for use and safekeeping in the library of the Cleveland Engineering Society.

"The book-stacks in the Society's library are being dismantled and removed to the United Engineering Library, where they will be re-erected as speedily as practicable. Certain reference books and the periodicals have been reserved for the reading-room in the Society's new quarters, to which they will be taken before the end of this month.

"Removal of the former roof of the Engineering Societies' Building and other operations immediately above the United Engineering Library necessitated the closing of the library for two periods of about one week each during the summer. Excepting these periods and the brief time occupied in moving from 57th Street, the books have been constantly accessible for use.

"By resolution of the Board of Direction, the ownership of that portion of the Society's library which was moved into the United Engineering Library, was formally tendered the United Engineering Society, but up to the date of this report, no formal acceptance had been received.

"No progress has been made on the physical part of the work of combining the four collections of books which now constitute the United Engineering Library, because of conditions created by the building operations and the fact that the much needed additional space and shelving have not been ready. Nevertheless, this matter, including the necessary recataloguing and indexing of the books, has been receiving the careful attention of Dr. Craver, the Director of the Library.

"Other matters affecting the library and the House of the Society are covered by the report of the Secretary.

"Respectfully submitted,

"George A. Harwood,
Lewis D. Rights,
J. V. Davies,
Charles Warren Hunt,
Alfred D. Flinn,
"Library Committee."

Alfred D. Flinn,
"Chairman."

Resolutions Adopted by the Executive Committee of the Board of Direction of the American Society of Civil Engineers, November 1st, 1917.

Whereas, Congress has passed an act known as the War Revenue Act of 1917; and

Whereas, Under Section 200 of said
trade and business include professions and occupations"; and

Whereas, Section 209 of said act states "That in the case of a trade or business having no invested capital or not more than a nominal capital there shall be levied, assessed, collected, and paid, in addition to the taxes under existing law and under this act, in lieu of the tax imposed by section two hundred and one, a tax equivalent to eight per centum of the net income of such trade or business, in excess of the following deductions: in the case of a domestic corporation, $3 000, and in the case of a domestic partnership, or a citizen or resident of the United States, $6 000, in the case of all other trades or business, no deduction"; and

Whereas, In most cases the professional man's earnings constitute his sole income, from which must be derived the capital which will yield a competency for himself and family, and inasmuch as these earnings are subject to the established income tax, including super-taxes, this same income should not be taxed further as if it were excess profits, when as a matter of fact it is not excess profits; and

Whereas, It is discriminatory and unjust to tax earned income at a higher rate than unearned income; and

Whereas, In other countries where incomes are taxed the tax is heavier on unearned than on earned incomes, thus encouraging industry and thrift;

Resolved, That in the opinion of the Executive Committee of the Board of Direction of the American Society of Civil Engineers, the provisions in Sections 200 and 209, hereinabove quoted, should be repealed because they are discriminatory and unjust.

Resolved, That the Secretary be and he is hereby directed to publish the above in *Proceedings,* and to send copies to the other National Societies, and to the various Associations of Members, urging similar action. Also, to forward copies to Hon. F. M. Simmons, Chairman of the Committee on Finance of the Senate, and Hon. Claude Kitchin, Chairman of the Ways and Means Committee of the House, requesting the repeal of the above-mentioned provisions at the next session of Congress. Also, to urge each member of the Society to act individually by communicating directly with his representatives in Congress along similar lines.

Remission of Dues

The Secretary has been authorized to inform the membership that remission of dues cannot be made in advance, but that all members of the Society who enter the United States Military Service will be carried on the rolls, if they so request, until the War is over.

"20 September 1917.

"To the Engineering Foundation Board

"Gentlemen: As this meeting terminates the year's agreement under which the Engineering Foundation has appropriated its income to the National Research Council, a brief summary of what has been accomplished seems called for.

"The Research Council was organized on the 19 September 1916. The Executive Committee was appointed at that meeting; since that date two other meetings of the Council have been held, one in Boston on 13-14 November, and the other in Washington on 19 April. During the intervals between the meetings of the Research Council, the Executive Committee has conducted all of the business of the Council. There have been twenty-nine meetings of the Executive Committee, thirteen in New York up to the first of May, fifteen in Washington subsequently, and one in Boston.

"The meetings in New York were devoted to the organization of the Research Council and especially to appointment of Committees. At the outset, the Council had no very clear idea of the way in which it was to organize; there were several other groups of scientific men at work, among them, the Research Committee of the American Association for the Advancement of Science and the Engineering Committee of the Advisory Commission to the Council of National Defense. Time was spent in straightening out these two principal complications, both of which were satisfactorily adjusted. The upshot of these negotiations is to leave the Research Council as the only representative, national body of research men, assisting the Government.

"During the period up to May, the Executive Committee in New York selected carefully the personnel of some twenty-five principal committees, and of a large number of sub-committees; so carefully were these chosen that of the entire number, several hundred, there have been only one or two declinations.

"Some of the important steps in the development of the Research Council are the establishment of the Military Section, composed of the Members of the Council connected with the Government and resident in Washington. This Section put the Research Council into close contact with the most important officials of the Army and Navy. From it has grown the Washington Office of the Council, and the recognition of the Research Council as the 'Department of Science and Research of the Council of National Defense,' and the establishment of the Washington Office after the Council of National Defense had officially recognized the Research Council. About this time Dr.

Hale, Chairman of the Council, appointed Dr. Millikan to be Vice-Chairman to represent the Council in Washington, and Dr. Millikan has since then been in charge of the work of the Council.

"At the request of the Secretary, Dr. Millikan has prepared a brief report of some of the work of the Research Council during its first year, which is appended to this report. It shows that the Research Council has been active in much of the important work under way in our preparations for war, and that it has rendered effective [illegible] of the Research Council—how it should best be formed and what scope it should have. In April, this was worked out by the Chairman, Mr. Dunn, most satisfactorily with the coöperation of the four principal Engineering Societies. This committee has since been one of the most active of the Research Council, under the efficient handling of Dr. Durand, who came on from California to act as head of the office in Washington and under whom the work of the Engineering Committee has been conducted. The work of the Secretary for the last few months has been principally with Dr. Durand and this Committee.

"The funds of the Engineering Foundation have been spent in general administrative work, that is to say, in the payment of traveling expenses, expenses of committees, stationery, printing, salaries, etc., of this office, principally. Money has not been spent directly in the solution of scientific problems.

"There is appended a brief financial statement, showing the operation of the Foundation for the year, resulting in a surplus of $4 775, which added to the surplus of $11 670 on 16 September 1916, makes a surplus at this date of $16 445. Against this the only liability is $1 000; and the Foundation has in this office furniture and fittings costing $838.

"The Engineering Foundation at its meeting in January approved a budget for the year amounting to $12 510 for the Research Council; subsequently, it authorized $400 more for furniture, in all, $12 910. The total amount disbursed on account of the Research Council to the 20 September 1917, is $10 445—that is, $2 465 less than the amount appropriated.

"The Foundation, in July, 1916, made an appropriation of $1 000 to Messrs. Marx and Durand of California for researches on wear of gears. The Secretary has recently written to Prof. Marx asking whether it would be possible, by the payment of expenses already incurred, to release the Foundation from subsequent expenses in this matter. An answer was requested by 20 September, but none has yet been received. The Foundation may be released from the obligation; but, for the present, it should be assumed that this $1 000 is tied up, leaving available for other purposes of the Foundation a surplus of $15 445.

"The Engineering Foundation has been an important factor in the formation of the National Research Council and in the guidance of its early life. It is, I think, safe to say, had it not been for the assistance given by the Foundation to the Research Council, that when its opportunity came to be of service to the country it would

not have been ready to act, and would not have been authorized as the 'Department of Science and Research of the Council of National Defense.'

"The Research Council no longer requires the financial assistance of the Foundation, which is, therefore, free to devote its resources to other work.

"Faithfully yours,

"CARY T. HUTCHINSON,
"Secretary."

FINANCIAL STATEMENT OF ENGINEERING FOUNDATION FOR THE YEAR, 29 SEPTEMBER 1916-1917.

"EXPENDITURES

"Account.	Amount.	
"Salary of Secretary	$5 000	
Traveling expenses	930	
Current office expenses (ex stationery)	2 133	
Office furniture and fittings	838	
Stationery (New York and Washington)	1 034	
Committee appropriations	510	
"Total expenditures		$10 445

"INCOME FOR THE YEAR

From securities (including bank interest)	10 220	
Mr. Swasey's gift	5 000	
"Total income		15 220
"Surplus for the year		
" on 16 September 1916		4 775
" on 20 September 1917		11 670
		16 445

"BUDGET

"Authorized for the Research Council	12 510	
" additional (for furniture)	400	
Total budget	12 910	
Expenditures (above)	10 445	
Remainder, not used		2 465

"THE ADAMS' FUND

"Gift of Mr. Adams	5 000	
Disbursements to 20 September 1917	1 597	
"Balance, 20 September 1917		3 403

"CARY T. HUTCHINSON,
"20 September 1917 *"Secretary."*

Dr. Millikan's Letter to Dr. Hutchinson

"September 7, 1917.

"Dr. Cary T. Hutchinson, Secretary,
The Engineering Foundation,
33 West 39th Street, New York.

"Dear Dr. Hutchinson: The following is a statement of some of the work of the National Research Council, condensed with difficulty on account of the great variety and scope of the Council's activities.

"All of the work of the Research Council that touches upon Army or Navy problems is carried on with the advice, co-operation, or control, as the case may be, of the representatives of the various Departments of the Army or Navy under which such work comes.

"The Council has co-operated in the establishment and organization of the submarine experimental work at Nahant and has also established a very active submarine station at New London, another at San Pedro, California, and has been instrumental in the organization of groups working at New York, Chicago, and Madison, Wisconsin.

"There has resulted a great practical advance in the art of submarine detection which it is not desirable to go into further.

"The Physics Committee of the Council has distributed to various groups twenty or more large problems in physics, which are being actively worked upon and some of which have already been solved. Among the latter are the location of aircraft by sound, the development of fire control for anti-aircraft guns, telephoning between airplanes, protection of balloons from ignition by static charges, and the development of new and improved methods of measuring muzzle velocities.

"The Chief Officer of the Signal Corps of the Army has asked the Research Council to act as the Division of Science and Research of the Signal Corps, and in this capacity the Council has organized a Sound Ranging Service in the Signal Corps, a new Meteorological Service in the Signal Corps, and is now drawing specifications for scientific instruments to be used on airplanes. It has sent a dozen of the best physicists in the country to France to aid the American Expeditionary Forces with their scientific knowledge, and is selecting a personnel of several hundred men who are to be engaged in the scientific services of the Army and Navy.

"The Chemistry Committee has perfected an elaborate organization for the handling of all the chemical problems which arise in the Army and in the Navy, and it has distributed some 150 chemical problems which are being attacked in the chemical laboratories of the country.

"The Psychology Committee has presented to the Secretary of War and the Adjutant General a vast program for the selection of officers for the Army from officers' reserve camps and for the classification of drafted men. In fact it has called in most of the best known psychologists of the country and has organized them and employment experts into a large group in whose hands the War Department has

placed the largest responsibilities regarding the examination and selection of men.

"The Medical Committee has enlisted the services of a large number of medical men of the country both in medical research problems and in the regular work of the Sanitary Corps of the Army.

"The Engineering Committee has contributed in no small degree to the development of devices for the protection of ships from submarines. It has organized a large group which are now working on the development of steel protective devices for use of the soldiers at the front, and through co-operation with the National Advisory Committee for Aeronautics it has carried on extensive and important researches in the development of airplanes and airplane engines.

"Turning to the work of the special committees of the Council, the Nitrate Committee has made an elaborate study and report which has been made the basis for the expenditure by the Government of large sums of money upon the erection of a nitrate plant.

"The Gas Warfare Committee has had for six months 120 chemists working on the problems of gas warfare and the results already attained have been of the utmost importance—so important that the Army and Navy have placed large appropriations at the disposal of this Committee for its researches.

"The Optical Glass Committee, by taking from the research laboratories like the Geophysical Laboratory, a dozen more silicate chemists and putting them directly in the works of the Bausch and Lomb Company and the Pittsburgh Plate Glass Company, has in six months' time developed in America the production of optical glass from nothing up to 20 000 pounds a month and in two months more this figure will have been multiplied two or three fold.

"The Psychiatry Committee has established abroad a Laboratory for the study of shell shock.

"The Foreign Service Committee, which the Council sent abroad at once upon the outbreak of the war, was wholly responsible for the sending back to this country of a French, English and Italian Scientific Mission, which brought with them the contributions which science had made to the war, both in the matter of instruments and methods, and unquestionably saved months of time in putting the United States abreast of the European situation, as regards modern scientific methods in warfare. It is difficult to overestimate the stimulus to American participation in the war which resulted directly from the action of the Research Council in sending abroad at once this Foreign Service Committee composed of seven of the best scientists in the country.

"These are a few of the results which have followed from the assistance which the Engineering Foundation gave in the bringing into being of the National Research Council. It is hoped that they are only a small part of the results which will have been attained by the end of the second year of its existence.

"Sincerely yours,

"R. A. Millikan,
"Vice-Chairman."

Reciprocal Resolutions Exchanged Between The Engineering Foundation and National Research Council

From the National Research Council to The Engineering Foundation:

"As the year for which the co-operative arrangement was made between the Engineering Foundation and the National Research Council will soon expire, the Research Council wishes again to record its high appreciation of the liberal financial and personal assistance which the Foundation and its members have afforded in the organization and support of the work of the Council in this first year of existence. It seems exceedingly desirable to maintain close relations between engineers and research workers which the Research Council has for the first time succeeded in establishing. Accordingly, the Council wishes to express the hope that it may be practicable to continue some effective form of co-operation between the two bodies, and it would be glad to appoint representatives to consider this matter with men designated by the Foundation."

From The Engineering Foundation to the National Research Council:

"*Resolved,* That The Engineering Foundation receives with pleasure the resolution of the National Research Council expressing appreciation of the financial and personal assistance rendered to the organization and work of the Council during the past year. Reciprocating the Council's desire for the maintenance of close relations between workers in science and in engineering, the Foundation hereby declares that it will be its policy to continue the co-operation between the two bodies in all practicable ways that may be now, or may become, mutually available, such ways consisting of the interchange of helpful suggestions, advice, information, office representation and similar facilities, and in addition a recognition of community of purpose that shall promote in the field of engineering research increasingly intimate relations between engineering and science.

"The reciprocal designation of the Secretary of each of the bodies as an assistant Secretary of the other for the purpose of enabling both bodies to have offices in both New York and Washington is favorably regarded, and the Foundation welcomes and gladly accepts the offered assistance of the Research Council in the national co-relation of the engineering research work of the Foundation to which the Foundation's resources will be devoted."

The Engineering Societies Club of Hawaii

The following letter from the Secretary of the Engineering Societies Club of Hawaii is presented for the information of the membership.

"Honolulu, Hawaii, October 9th, 1917.

"Mr. Charles Warren Hunt,

Secretary of the American Society of Civil Engineers.

"Dear Sir: Kindly be informed that The Engineering Societies Club of Hawaii has been established, and I am herewith enclosing a

copy of the 'Constitution and By-Laws' which will give you the purposes of this Club. As you will note it is an amalgamation of the:

American Society of Civil Engineers,
American Society of Mechanical Engineers,
American Institute of Mining Engineers,
American Society of Naval Engineers,
American Railway Engineering Association,
American Society for Testing Materials,

a member of any of these national organizations being eligible to become a member of the Club.

"The main purpose of this Club is for the entertainment of visitors from the various National organizations while in Honolulu and the Hawaiian Islands. We therefore request that the various national organizations post notices of the existence of this Club in their various magazines and transactions so that all members of these national organizations will know that such a Club exists and the privileges extended to them while visiting Honolulu.

"We trust that the various Secretaries who are aware of a proposed visit to the Islands by any one of its members would communicate with the Secretary of our Club so that he may be taken care of in the proper manner. We would wish also that any member proposing to visit Honolulu or the Islands would make it a point to announce himself to the Secretary of The Engineering Societies Club of Hawaii, so that the various members may get into communication with him.

"Trusting that due notice will be given the various members of your national organizations of the existence of this Club, and its purposes, thanking you,

"Very truly yours,
"THE ENGINEERING SOCIETIES CLUB OF HAWAII,
"By E. F. CYKLER,
"Secretary."

ANNOUNCEMENTS

The Reading Room of the Society is open from 9 A. M. to 10 P. M., every day, execpt Sundays, Fourth of July, Thanksgiving Day, and Christmas Day.

FUTURE MEETINGS

December 5th, 1917.—8.30 P. M.—Chas. Warren Hunt, Sec., Am. Soc. C. E., will address the meeting on Society matters of general interest.

December 19th, 1917.—8.30 P. M.—At this meeting a paper entitled "Construction Problems of the Manhattan-Bronx and Lexington Avenue Subway Junction and Queensborough Tunnel Connections," by George Perrine, M. Am. Soc. C. E., will be presented for discussion. This paper is printed in this number of *Proceedings*.

ANNUAL MEETING

The Sixty-fifth Annual Meeting will be held at the Headquarters of the Society, 33 West 39th Street, on Wednesday and Thursday, January 16th and 17th, 1918. The Business Meeting will be called to order at 10 o'clock on Wednesday morning. The Annual Reports will be presented, Officers for the ensuing year elected, members of the Nominating Committee appointed, Reports of Special Committees presented for discussion, and other business transacted.

SEARCHES IN THE LIBRARY

In January, 1902, the Secretary was authorized to make searches in the Library, upon request, and to charge therefor the actual cost to the Society for the extra work required. Since that time many searches have been made, and bibliographies and other information on special subjects furnished.

The resulting satisfaction, to the members who have made use of the resources of the Society in this manner, has been expressed frequently, and leaves little doubt that if it were generally known to the membership that such work would be undertaken, many would avail themselves of it.

The cost is trifling compared with the value of the time of an engineer who looks up such matters himself, and the work can be performed quite as well, and much more quickly, by persons familiar with the Library.

In asking that such work be undertaken, members should specify clearly the subject to be covered, and whether references to general books only are desired, or whether a complete bibliography, involving search through periodical literature, is desired.

It sometimes happens that references are found which are not readily accessible to the person for whom the search is made. In that case the material may be reproduced by photography, and this can be done for members at the cost of the work to the Society, which is small. This method is particularly useful when there are drawings or figures in the text which would be very expensive to reproduce by hand.

A list of 989 bibliographies made in the Library, giving the cost of each, was published in Vol. LXXX of *Transactions.*

Since October 1st, 1916, the Library of the American Society of Civil Engineers has ceased to exist, as such, having been merged with the Libraries of the Mining, Mechanical, and Electrical Engineers, and become a part of the Library of the United Engineering Society. There were 67 000 accessions, which were not duplicates, turned over to that Library.

Hereafter, therefore, requests for searches should be addressed to the Librarian, United Engineering Society, 29 West 39th Street, New York City.

PAPERS AND DISCUSSIONS

Members and others who take part in the oral discussions of the papers presented are urged to revise their remarks promptly. Written communications from those who cannot attend the meetings should be sent in at the earliest possible date after the issue of a paper in *Proceedings.*

All papers accepted by the Publication Committee are classified by the Committee with respect to their availability for discussion at meetings.

Papers which, from their general nature, appear to be of a character suitable for oral discussion will be published as heretofore in *Proceedings,* and set down for presentation to a future meeting of the Society, and on these oral discussions, as well as written communications, will be solicited.

All papers which do not come under this heading, that is to say, those which from their mathematical or technical nature, in the opinion of the Committee, are not adapted to oral discussion, will not be scheduled for presentation to any meeting. Such papers will be published in *Proceedings* in the same manner as those which are to be presented at meetings, but written discussions only will be requested for subsequent publication in *Proceedings* and with the paper in the volumes of *Transactions.*

The Board of Direction has adopted rules for the preparation and presentation of papers, which will be found on page 429 of the August, 1913, *Proceedings.*

LOCAL ASSOCIATIONS OF MEMBERS OF THE AMERICAN SOCIETY OF CIVIL ENGINEERS

San Francisco Association, Organized 1905.

J. D. Galloway, President; E. T. Thurston, Secretary-Treasurer, 57 Post Street, San Francisco, Cal.

The San Francisco Association of Members of the American Society of Civil Engineers holds regular bi-monthly meetings, with banquet, and weekly informal luncheons. The former are held at 6. P. M., at the Palace Hotel, on the third Tuesday of February, April, June, August, October, and December, the last being the Annual Meeting of the Association.

Informal luncheons are held at 12.30 P. M., every Wednesday, and the place of meeting may be ascertained by communicating with the Secretary.

The by-laws of the Association provide for the extension of hospitality to any member of the Society who may be temporarily in San Francisco, and any such member will be gladly welcomed as a guest.

(Abstract of Minutes of Meeting)

October 16th, 1917.—The meeting was called to order at the Palace Hotel; President Galloway in the chair; Nathan A. Bowers, Secretary, *pro tem.*; and present, also, 53 members and guests.

The Entertainment Committee provided several reels of moving pictures, three of which were supplied by the Trojan Powder Company and the Oakland Paving Company.

Messrs. Newman, Cleary, and Chew were appointed as the Entertainment Committee for the Annual Meeting in December.

For the Committee on Military Affairs, President Galloway reported that the Committee had completed its work and had adjourned *sine die.* Of the $750 contributed to this Committee for its work, about $135 was not used, of which $47 had been contributed to relief work in France and the remainder to the work of the Red Cross.

President Galloway reported that the bound volumes of *Transactions* of the Society (donated by Mr. Schulze as prizes for essays written by Juniors) had been awarded to Messrs. Bolin and Bithers for a paper entitled "The Sewers of Berkeley", which would be presented before the Association at a later date.

Attention was called to the joint meeting of the Local Sections of the American Society of Mechanical Engineers, the American Institute of Mining Engineers, the American Institute of Electrical Engineers, the American Chemical Society, and the American Society of Civil Engineers, to be held on October 25th, 1917, in honor of Dr. Ira N. Hollis, President of the American Society of Mechanical Engineers.

A paper by Mr. M. M. O'Shaughnessy, entitled "The Tunnels of San Francisco", was presented by the author.

President Galloway presented a letter from Joseph Jacobs, President of the Seattle Association, in which Mr. Jacobs suggested changes in the status of the Local Associations.

A letter was also read from the Connecticut Society of Civil Engineers suggesting the advisability of fostering among college students

the formation of some undergraduate Society, which would lead the men to consideration of affiliation with the Society.

A letter was read from Mr. E. T. Thurston, now at the Engineers' Training Camp, Vancouver Barracks, Wash., in which he reported rapid progress being made at the Camp.

Mr. Bowers reported having visited the new Headquarters of the Society in the Engineering Societies Building, in New York City, and

Colorado Association, Organized 1908.

Robert Follansbee, President; L. R. Hinman, Secretary-Treasurer, 1400 West Colfax Avenue, Denver, Colo.

The meetings of the Colorado Association of Members of the American Society of Civil Engineers (Denver, Colo.) are held on the second Saturday of each month, except July and August. The hour and place of meeting are not fixed, but this information will be furnished on application to the Secretary. The meetings are usually preceded by an informal dinner. Members of the American Society of Civil Engineers will be welcomed at these meetings.

Weekly luncheons are held on Wednesdays at 12.30 P. M., at Daniel's and Fisher's.

Visiting members are urged to attend the meetings and luncheons.

Atlanta Association, Organized 1912.

Paul H. Norcross, President; Thomas P. Branch, Secretary-Treasurer, Georgia School of Technology, Atlanta, Ga.

The Association holds its meetings at the University Club, Atlanta, Ga. Regular monthly luncheon meetings are held to which visiting members of the Society are always welcome.

Baltimore Association, Organized 1914.

Mason D. Pratt, President; Charles J. Tilden, Secretary-Treasurer, The Johns Hopkins University, Baltimore, Md.

Cleveland Association, Organized 1914.

W. J. Watson, President; George H. Tinker, Secretary-Treasurer, 516 Columbia Building, Cleveland, Ohio.

Detroit Association, Organized 1916.

T. A. Leisen, President; Clarence W. Hubbell, Secretary, 2348 Penobscot Building, Detroit, Mich.

The regular meetings of the Association are held on the second Friday of December, April, and October, the last being the Annual Meeting.

District of Columbia Association, Organized 1916.

A. P. Davis, President; John C. Hoyt, Secretary-Treasurer, U. S. Geological Survey, Washington, D. C.

Duluth Association, Organized 1917.

F. E. House, President; Walter G. Zimmermann, Secretary, Wolvin Building, Duluth, Minn.

The regular meetings of the Association are held at noon on the third Monday of each month (usually at the Kitchi Gamma Club), with luncheon, followed by a short business session and reading of papers. Visiting members of the American Society of Civil Engineers can secure from the Secretary definite information relating to the meetings, at which they will be welcomed. The Annual Meeting is held on

(Abstract of Minutes of Meeting)

October 15th, 1917.—The meeting was called to order; Vice-President Darling in the chair; Walter G. Zimmermann, Secretary; and present, also, 21 members and 1 guest.

The resignation of Mr. E. W. Kelly as Treasurer was accepted, he having received a commission as a Captain in the United States Engineers Reserve. Mr. John Carson was elected to fill the vacancy.

Mr. John H. Darling was unanimously suggested by the Association as member of the Nominating Committee to be chosen from District No. 7.

A letter was read from Mr. Edward P. Burch, of Minneapolis, Minn., inviting the Association to join with other Engineering Societies of the State in a dinner to Mr. John R. Allen, the new Dean of the College of Engineering and Architecture of the University of Minnesota, on October 18th, 1917. Mr. W. H. Hoyt was appointed the special representative of the Association at this dinner.

A discussion of the paper entitled "Soundings", which had been presented by Mr. O. H. Dickerson at a previous meeting, was continued, many of the members present taking part in it.

Adjourned.

Illinois Association, Organized 1916.

C. F. Loweth, President, Chicago, Ill.

The regular meetings of the Association are held on the second Monday of March, June, September, and December, the last being the Annual Meeting. The hour and place of meeting are not fixed, but this information will be furnished on application to the President.

Louisiana Association, Organized 1914.

W. B. Gregory, President; Charles W. Okey, Secretary, Tulane University, New Orleans, La.

The regular meetings of the Association are held at The Cabildo, New Orleans, La., on the first Monday of January, April, July, and October.

Nebraska Association, Organized 1917.

Frank T. Darrow, President; Homer V. Knouse, Secretary-Treasurer, 115 City Hall, Omaha, Nebr.

Regular meetings of the Association are held on the first Saturday of each month, except July and August, and at such places as may be appointed from time to time by the Executive Committee. The Annual Meeting is held in Lincoln, Nebr., on the second Friday in January.

It is probable that frequent luncheons will be held in Omaha, in addition to the monthly meetings, at which visiting members will be

welcomed. The place of meeting may be ascertained by communicating with the Secretary.

Northwestern Association, Organized 1914.

George L. Wilson, President; Ralph D. Thomas, Secretary, 508 South First Street, Minneapolis, Minn.

Samuel T. Wagner, President; C. W. Thorn, Secretary, 1313 South Broad Street, Philadelphia, Pa.

The regular meetings of the Association are held at the Engineers' Club of Philadelphia, 1317 Spruce Street, on the First Monday in January, April, and October, the last being the Annual Meeting.

Portland, Ore., Association, Organized 1913.

J. P. Newell, President; J. A. Currey, Secretary, 194 North 13th Street, Portland, Ore.

St. Louis Association, Organized 1888 (1914).

J. A. Ockerson, President; C. M. Daily, Secretary-Treasurer, 4240 Shaw Avenue, St. Louis, Mo.

The Annual Meeting of the Association is held on the fourth Monday in November in the Auditorium of the Engineers Club of St. Louis. The time and place of other meetings are not fixed, but this information will be furnished on application to the Secretary.

(Abstract of Minutes of Meetings)

September 10th, 1917.—The meeting was called to order at 12.30 P. M., at the City Club; President Ockerson in the chair; Cornelius M. Daily (Secretary); and present, also, 16 members.

The minutes of the preceding meeting were read and approved.

President Ockerson read extracts from the minutes of the meeting of the Board of Direction in regard to correcting a statement in the May *Proceedings* as to the date of organization of the Association.

On motion, duly seconded, the following resolution was passed unanimously:

"*Whereas:* A local association of the St. Louis members of the American Society of Civil Engineers was organized in February, 1888, at which time a permanent Chairman, Vice-Chairman, and Secretary were elected, the names of the charter members being:

Robert Moore	R. Hering	B. L. Crosby
John A. Ockerson	A. M. Wellington	O. B. Wheeler
George A. Marr	W. Bartlett	Geo. H. Pegram
Edw. C. Rice	N. W. Eayrs	M. L. Holman
J. B. Johnson	J. Baler	——Parkhurst
Edw. Flad	——Blaisdell	Fred Brooks
C. V. Mersereau	R. E. McMath	T. J. Long
S. B. Russell	Carl Gaylor	Geo. Brant
		E. L. Meier

and

"*Whereas:* The said Association adopted a Constitution and By-laws which in their main features are similar to the Constitution and

By-laws approved by the American Society of Civil Engineers in 1914; therefore,

"Be it Resolved: That the Board of Direction be and is hereby requested to direct that under the general heading in the *Proceedings*, 'Local Associations of Members of the American Society of Civil Engineers', the date of organization of the St. Louis Association be hereafter stated as 1888 instead of 1914, adding the date of admission to

On motion, duly seconded, it was decided that the meetings for the coming year shall be held alternately mid-day and evening.

On motion, duly seconded, the Chairman was authorized to appoint a Nominating Committee to nominate officers for the annual election in November.

President Ockerson appointed a committee consisting of Messrs. Rolfe, Woermann, and Daily to arrange for meetings.

The Secretary was instructed to forward a copy of the resolutions to the Board of Direction and also ask for correction of the statement in the *Proceedings* relative to the place for holding meetings.

Adjourned.

October 5th, 1917.—The meeting was called to order at 12.30 P. M.; President Ockerson in the chair; Cornelius M. Daily, Secretary.

A motion that the Association, as evidence of its interest in all students of Civil Engineering, endorse the proposed amendment to the Constitution, permitting the formation of Student Branches, was carried; and the Secretary was instructed to inform the Committee, appointed by the Board of Direction to investigate the merit of the proposed amendment, and also the Connecticut Society, of the action of the Association in this matter.

Mr. Baxter L. Brown, of St. Louis, was the unanimous choice for the Nominating Committee from District No. 9.

On motion, duly seconded, a circular letter was ordered to be sent to all corporate members in District No. 9, requesting them to endorse Mr. Brown for the Nominating Committee.

Adjourned.

San Diego Association, Organized 1915.

W. J. Gough, President; J. R. Comly, Secretary-Treasurer, 4105 Falcon Street, San Diego, Cal.

Seattle Association, Organized 1913.

Joseph Jacobs, President; Carl H. Reeves, Secretary-Treasurer, 444 Henry Building, Seattle, Wash.

The regular meetings of the Association are held at 12.15 P. M., on the last Monday of each month, at The Frye Hotel.

Southern California Association, Organized 1914.

H. Hawgood, President; H. W. Dennis, Secretary, 329 San Fernando Building, Los Angeles, Cal.

The Southern California Association of Members of the American Society of Civil Engineers (Los Angeles, Cal.) holds regular bi-monthly

meetings with banquet, at Hotel Clark, on the second Wednesday of February, April, June, August, October, and December, the last being the Annual Meeting of the Association.

Informal luncheons are held at 12.15 P. M. every Wednesday, and the place of meeting may be ascertained from the Secretary.

The by-laws of the Association provide for the extension of hospitality to any member of the Society who may be temporarily in Los Angeles, and any such member will be gladly welcomed as a guest at any of the meetings or luncheons.

(Abstract of Minutes of Meeting)

October 10th, 1917.—The meeting was called to order at 6 P. M., at the Hotel Clark; President Hawgood in the chair; and W. K. Barnard, acting as Secretary.

The reading of the minutes of the previous meeting was dispensed with.

President Hawgood announced the resignations of Mr. Charles H. Lee, as Treasurer, and Mr. Wilkie Woodard, as Secretary, of the Association, both having resigned in order to join the colors as officers in the Engineers' Reserve Corps.

The President announced that the Board of Directors had appointed Mr. H. W. Dennis to succeed Mr. Woodard as Secretary and Mr. Ralph J. Reed to succeed Mr. Lee as Treasurer for the unexpired portions of their terms of office.

Messrs. A. L. Sonderegger and C. C. Huff were appointed a Committee to audit the books of the Association before they were turned over to Mr. Reed.

A letter was read from the City Planning Association of Los Angeles inviting the Association to name five members who might be willing to co-operate with it in working out city planning projects.

A letter was read from Henry R. Buck, President of the Connecticut Society of Civil Engineers, relative to the formation of Student Branches of the Parent Society.

On motion, duly seconded, this movement was endorsed.

A letter from Joseph Jacobs, President of the Seattle Association, was read, bearing on the subject of the proper relation of the Parent Society to the Local Associations. Mr. Jacobs also enclosed a copy of a letter on the subject, which he had forwarded to the Society.

The Secretary was instructed to send copies of these communications to all members of the Association for their study, in order that discussion might be had thereon at the Annual Meeting and a definite report be made to the Parent Society immediately afterward.

Communications were read from Mr. H. F. Withey enclosing correspondence from Professor C. W. Cook, relative to engineering work being done by the University of California.

The President appointed Mr. Reed a committee to investigate the matter of engineering work and to determine further whether similar work was being done by other educational institutions in this territory.

The matter of subscription to the second Liberty Loan was announced by President Hawgood, who stated that information on the subject would be given to any member of the Association requesting it.

Mr. George A. Damon, Vice-President of the Joint Committee of

the Technical Societies of Los Angeles and Chairman of a Special Committee to formulate a plan for the presentation to the public of the engineering fundamentals of municipal and other public problems, addressed the meeting regarding the work of such a Committee. Mr. Damon's address was followed by a discussion on the subject by those present.

Adjourned.

Spokane Association, Organized 1914.

J. C. Ralston, President; B. J. Garnett, Secretary, City Hall, Spokane, Wash.

The regular meetings of the Association are held on the second Friday of each month, except July and August. The hour and place of meeting are not fixed, but this information will be furnished on application to the Secretary.

Visiting members are invited to attend the meetings and luncheons.

Texas Association, Organized 1913.

John B. Hawley, President; J. F. Witt, Secretary, Dallas, Tex.

Utah Association, Organized 1916.

George L. Swendsen, President; H. S. Kleinschmidt, Secretary-Treasurer, 306 Dooly Building, Salt Lake City, Utah.

The Annual Meeting of the Association is held on the first Wednesday in April. The time of other meetings is not fixed, but this information will be furnished on application to the Secretary.

MINUTES OF MEETINGS OF SPECIAL COMMITTEES TO REPORT UPON ENGINEERING SUBJECTS

Special Committee on Materials for Road Construction

October 15th, 1917.—The meeting was called to order at 9.45 A. M., at the House of the Society. Present, George W. Tillson (Chairman *pro tem.*), H. K. Bishop, A. W. Dean, Nelson P. Lewis, and A. H. Blanchard (Secretary).

The minutes of the meeting of August 17th, 1917, were approved.

The final reports of the Sub-Committees on "Introduction and General Principles" and "Subjects for Investigation", were discussed at length, and several amendments relative thereto were adopted.

October 22d, 1917.—The meeting was called to order at the House of the Society at 10 A. M. Present, George W. Tillson (Chairman *pro tem.*), H. K. Bishop, Nelson P. Lewis, and Arthur H. Blanchard (Secretary).

The minutes of the meeting of October 15th, 1916, were approved.

The final reports of the Sub-Committees on Introduction and General Principles, Subjects for Investigation, Asphalt Block Pavements, Bituminous Concrete Pavements, Bituminous Macadam Pavements, Bituminous Surface Treatments, Brick Pavements, Broken Stone Roads, Earth and Sand-Clay Roads, Gravel Roads, Sheet Asphalt Pavements, Stone Block Pavements, and Wood Block Pavements, were discussed and adopted as amended.

The final report of the Sub-Committee on "Forms" was discussed, and several amendments relative thereto were adopted.

On motion, duly seconded, the Committee adjourned to meet at 9.30 A. M., on Saturday, October 27th, 1917, at the Engineering Societies Building.

October 27th, 1917.—The meeting was called to order at 9.40 A. M., in the Engineering Societies Building. Present, George W. Tillson (Chairman *pro tem.*), A. W. Dean, Nelson P. Lewis, Charles J. Tilden, and Arthur H. Blanchard (Secretary).

The minutes of the meeting of October 22d, 1917, were approved.

The final reports of the Sub-Committees on the following sections of the 1918 Report, were discussed at length and adopted as amended: Forms, Cement-Concrete Pavements, Analyses and Tests of Non-Bituminous Materials, and Analyses and Tests of Bituminous Materials.

On motion, duly seconded, the 1918 Report was adopted *en bloc*.

On motion, duly seconded, the Committee adjourned *sine die*.

Special Committee on Stresses in Railroad Track

October 16th-17th, 1917.—The meeting was called to order in Chicago, Ill., two sessions being held on October 16th, and one on October 17th. Present, Messrs. Talbot (Chairman), Baldwin, Bremner, Bronson (representing Dudley), Brunner, Burton, Churchill, Cushing, Dawley, Gennet and Morgan (representing Hunt), Jackson (representing Stimson), Jenkins, La Bach, Larsson, McGuigan (representing Safford), Ray, Turneaure, Willoughby, H. R. Thomas, Assistant Engineer of Tests, and Fritch (Secretary).

The tentative draft of a Progress Report of the Committee, which had been sent to the members on September 15th, 1917, and additional matter distributed later and at this meeting, were discussed in detail and modifications made.

The Chairman was authorized to prepare the Introductory Matter and to present the report to the American Society of Civil Engineers and to the American Railway Engineering Association as a Progress Report of the Committee.

It was voted to ask members not present at the meeting or not represented whether they approved the proposed Progress Report. Plans for future work were also discussed.

Special Committee on Steel Columns and Struts

October 22d, 1917.—The meeting was called to order at the House of the Society at 8.15 P. M. Present, Lewis D. Rights (Chairman), James H. Edwards, Ralph Modjeski, George F. Swain, and C. W. Hudson (Secretary). Professor Nelson, of the Bureau of Standards, was also present.

The minutes of the previous meeting were read and approved.

On motion, duly seconded, it was decided that the Committee would prepare a final report and ask to be discharged.

The Chairman presented a rough draft of a final report, which was discussed, and many suggestions made by those present were incorporated in it.

On motion, duly seconded, the Chairman was instructed to revise the report and submit it to a meeting to be called later by the Chair.

The diagrams and tables submitted with the report were approved by the Committee, and the Chairman was requested to confer with the Secretary of the Society relative to having these prepared for publication in advance of the completion of the report.

PRIVILEGES OF ENGINEERING SOCIETIES EXTENDED TO MEMBERS OF THE AMERICAN SOCIETY OF CIVIL ENGINEERS

Members of the American Society of Civil Engineers will be welcomed by the following Engineering Societies, both to the use of their Reading Rooms, and at all meetings:

American Institute of Electrical Engineers, 25 West Thirty-ninth Street, New York City.

American Institute of Mining Engineers, 25 West Thirty-ninth Street, New York City.

American Society of Mechanical Engineers, 25 West Thirty-ninth Street, New York City.

Assoçiacão dos Engenheiros Civis Portuguezes, Lisbon, Portugal.

Australasian Institute of Mining Engineers, Melbourne, Victoria, Australia.

Boston Society of Civil Engineers, 715 Tremont Temple, Boston, Mass.

Brooklyn Engineers' Club, 117 Remsen Street, Brooklyn, N. Y.

Canadian Society of Civil Engineers, 176 Mansfield Street, Montreal, Que., Canada.

Civil Engineers' Society of St. Paul, St. Paul, Minn.

Cleveland Engineering Society, Chamber of Commerce Building, Cleveland, Ohio.

Cleveland Institute of Engineers, Middlesbrough, England.

Dansk Ingeniorforening, Amaliegade 38, Copenhagen, Denmark.

Detroit Engineering Society, 46 Grand River Avenue, West, Detroit, Mich.

Engineering Association of Nashville, Commercial Club Building, Nashville, Tenn.

Engineers and Architects Club of Louisville, 1412 Starks Building, Louisville, Ky.

Engineers' Club of Baltimore, 6 West Eager Street, Baltimore, Md.

Engineers' Club of Kansas City, E. B. Murray, Secretary, 920 Walnut Street, Kansas City, Mo.

Engineers' Club of Minneapolis, 17 South Sixth Street, Minneapolis, Minn.

Engineers' Club of Philadelphia, 1317 Spruce Street, Philadelphia, Pa.

Engineers' Club of St. Louis, 3817 Olive Street, St. Louis, Mo.

Engineers' Club of Toronto, 96 King Street, West, Toronto, Ont., Canada.

Engineers' Club of Trenton, Trent Theatre Building, 12 North Warren Street, Trenton, N. J.

Engineers' Society of Northeastern Pennsylvania, 415 Washington Avenue, Scranton, Pa.

Engineers' Society of Pennsylvania, 31 South Front Street, Harrisburg, Pa.

Engineers' Society of Western Pennsylvania, 568 Union Arcade Building, Pittsburgh, Pa.

Florida Engineering Society, J. R. Benton, Secretary, Gainesville, Fla.

Institute of Marine Engineers, The Minories, Tower Hill, London, E., England.

Institution of Civil Engineers, Great George Street, Westminster, S. W., London, England.

Institution of Engineers of the River Plate, Calle 25 de Mayo 195, Buenos Aires, Argentine Republic.

Institution of Naval Architects, 5 Adelphi Terrace, London, W. C., England.

Junior Institution of Engineers, 39 Victoria Street, Westminster, S. W., London, England.

Koninklijk Instituut van Ingenieurs, The Hague, The Netherlands.

Louisiana Engineering Society, State Museum Building, Chartres and St. Ann Streets, New Orleans, La.

Memphis Engineers' Club, Memphis, Tenn.

Midland Institute of Mining, Civil and Mechanical Engineers, Sheffield, England.

Montana Society of Engineers, Butte, Mont.

North of England Institute of Mining and Mechanical Engineers, Newcastle-upon-Tyne, England.

Oregon Society of Civil Engineers, Portland, Ore.

Pacific Northwest Society of Engineers, 803 Central Building, Seattle, Wash.

Rochester Engineering Society, Rochester, N. Y.

Sociedad Colombiana de Ingenieros, Bogota, Colombia.

Sociedad de Ingenieros del Peru, Lima, Peru.

Societe des Ingenieurs Civils de France, 19 rue Blanche, Paris, France.

Society of Engineers, 17 Victoria Street, Westminster, S. W., London, England.

Svenska Teknologforeningen, Brunkebergstorg 18, Stockholm, Sweden.

Tekniske Forening, Vestre Boulevard 18–1, Copenhagen, Denmark.

Vermont Society of Engineers, George A. Reed, Secretary, Montpelier, Vt.

Western Society of Engineers, 1735 Monadnock Block, Chicago, Ill.

ACCESSIONS TO THE UNITED ENGINEERING SOCIETY LIBRARY

(From October 2d to November 1st, 1917)

DONATIONS*

The statements made in these notices are taken directly from the books themselves, and this Society is not responsible for them.

ELECTRICAL ENGINEERING:

The Theory and Characteristics of Electrical Circuits and Machinery. By Clarence V. Christie. 2d ed., rev. and enl. N. Y., McGraw-Hill Book Co., Inc.; Lond., Hill Publishing Co., Ltd., 1917. 20 + 546 pp., 508 illus., 9 x 6 in., cloth. $4.00.

Much new material has been added to this edition, the most important additions being sections on complex alternating waves and wave analysis, on polyphase alternating-current circuits, the construction of the characteristic curves of direct-current generators and motors, the design of direct and alternating-current machinery, the Blondel diagram for the synchronous motor, the symbolic method of analysis of the induction motor, alternating-current commutator motors, and a chapter on measuring instruments.

A TREATISE ON THE ELEMENTS OF ELECTRICAL ENGINEERING:

A Text Book for Colleges and Technical Schools. By William S. Franklin. Vol. 1, Direct and Alternating Current Machines and Systems. N. Y., The Macmillan Company, 1917. 465 pp., 368 illus., 9 x 6 in., cloth. $4.50. (Gift of the author.)

Written to encourage a first course in electrical engineering covering both direct and alternating-current machines and systems. The emphasis throughout is on the simple physics of the subject and on operating engineering. The book entitled "Electric Lighting and Miscellaneous Applications of Electricity" will henceforth be known as Vol. 2 of the present treatise.

PRACTICAL ELECTRIC ILLUMINATION.

By Terrell Croft. N. Y., McGraw-Hill Book Co., Inc.; Lond., Hill Publishing Co., Ltd., 1917. 11 + 225 pp., 166 illus., 8 x 6 in., cloth. $2.00.

Contains a concise account of the principles of illumination, the methods used and the proper ways of lighting indoor and outdoor spaces used for specific purposes. Contents: Preface; Fundamental Ideas of Light Radiation; Principles and Units; Reflectors; Incandescent Lamps; Arc Lamps; Nernst, Mercury-Vapor and Tube Lamps; Principles of Illumination Design; Interior Illumination; Exterior Illumination.

X-RAYS.

By G. W. C. Kaye. 2d ed. N. Y. and Lond., Longmans, Green, and Co., 1917. 17 + 285 pp., 115 illus., 9 x 6 in., cloth. $3.00.

The author of this book is the head of the Radium and X-Ray Department of the National Physical Laboratory. The subject-matter gives an account of such of the present-day methods and apparatus as appear valuable or novel, with a critical treatment of some of the features which have claimed the author's interest. The development of theory is discussed, and a summary of the history of the X-ray, from 1895 to the middle of 1916, is included.

* Unless otherwise specified, books in this list have been donated by the publishers.

ELEMENTS OF MACHINE DESIGN.

By O. A. Leutwiler. N. Y., McGraw-Hill Book Co., Inc.; Lond., Hill Publishing Co., Ltd., 1917. 15 + 607 pp., 339 illus., 9 x 6 in., cloth. $4.00.

Written to present in fairly completed form a discussion of the fundamental principles involved in the design and operation of machinery, and to outline or suggest methods of reasoning which may prove helpful in the design of various parts. Brief reviews are included of the more important straining actions to which machine parts are subjected and of the properties of the common materials used in machine construction, and references to additional sources of information accompany the various chapters.

MECHANICAL DRAWING PROBLEMS.

By Charles William Weick. N. Y., McGraw-Hill Book Co., Inc.; Lond., Hill Publishing Co., Ltd., 1917. 9 + 153 pp., 120 illus., 1 pl., 9 x 6 in., cloth. $1.25.

A book of examples and problems intended to be used under the direction of a teacher. It aims to provide a large number of typical drawings carefully worked out as examples, accompanied by appropriate problems. These vary in difficulty sufficiently to enable the teacher to formulate suitable courses for all schools where mechanical drawing is taught.

STEAM BOILERS:

A Practical and Authoritative Discussion of Boiler Design and Construction, and the Development of Modern Types. Revised by Robert H. Kuss. Chicago, American Technical Society, 1917. 82 pp., 74 illus., 8 x 6 in., cloth. $1.00.

Covers in a practical manner the methods of construction, the proper testing of materials, and the methods of riveting and staying. Describes different types of stationary and marine boilers.

STEAM CHARTS

And Special Tables for Turbine Calculations. By F. O. Ellenwood. N. Y., John Wiley & Sons, Inc.; Lond., Chapman & Hall, Ltd., 1917. 49 pp., 18 charts. $1.00.

Comprises a series of charts and tables to assist engineers in making calculations involving wet or superheated steam. In the present edition, intended for those thoroughly conversant with the application of the tables, the problems and explanatory matter have been eliminated. Tables giving the theoretical velocities of steam for each tenth of a heat unit up to 80 British thermal units, and the squares of numbers from 200 to 2 000 have been added.

LUBRICATING ENGINEERS HANDBOOK:

A Reference Book of Data, Tables, and General Information for the Use of Lubricating Engineers, Oil Salesmen, Operating Engineers, Mill and Power Plant Superintendents and Machinery Designers, etc. By John Rome Battle. Phila. and Lond., J. B. Lippincott Company (copyright 1917). 333 pp., 114 illus., 9 x 6 in., cloth. $4.00.

Describes the properties of various lubricants and the methods of testing them, and discusses the proper methods of lubricating steam and gas engines, refrigerating machinery, railway cars, locomotives, rolling mills, pneumatic tools, etc.

THE PRINCIPLES OF AEROGRAPHY.

By Alexander McAdie. N. Y. and Chic., Rand McNally Co. (copyright 1917). 12 + 318 pp., 112 illus., 8 x 6 in., cloth. $3.00.

The author's especial aim has been to give prominence to the results of recent aërological investigations. The centimeter-gram-second system of units is used. More than usual attention is given to cloud forms and the thermodynamics of their formation and dissipation.

AEROPLANE DESIGN.

By F. S. Barnwell, and "A Simple Explanation of Inherent Stability", by W. H. Sayers. N. Y., Robert M. McBride & Co., 1917. 102 pp., 28 illus., 7 x 5 in., cloth. $1.00.

These two papers have been published in *The Aeroplane.* In the first, Mr. Barnwell attempts *a précis* of the general principles for the information of trained engineers without experience in airplane design. Mr. Sayers offers a non-mathematical explanation of the known principles by which inherent stability may be attained.

LEARNING TO FLY IN THE U. S. ARMY:

A Manual of Aviation Practice. By E. N. Fales. N. Y., McGraw-Hill Book Co., Inc.; Lond., Hill Publishing Co., Ltd., 1917. 9 + 180 pp., 39 illus., 1 pl., 7 x 5 in., leather. $1.50.

Parallels the instruction given under the author's direction in the U. S. Ground School of Military Aeronautics, University of Illinois Branch. Sets forth for the non-technical student aviator what he must know in order properly to understand his airplane, to keep it trued up, and to operate it in cross-country flights.

MACHINE GUNS:

Part 1, Mechanism, by Capt. Julian S. Hatcher; Part 2, The Practical Handling of Machine Gun Fire, by First Lt. Glenn P. Wilhelm; Part 3, Machine Gun Tactics, by First Lt. Harry J. Malony. Menasha, George Banta Publishing Company (copyright 1917). 233 pp., 84 illus., 3 diagrams, 8 x 5 in., cloth. $2.50.

The authors have been instructors in the Machine Gun School, at Harlingen, Tex., since its establishment in 1916, and have based this manual on notes made during their work. The guns described are the Benet-Mercié, the Lewis, the Maxim, the Vickers, and the Colt.

APPLIED ELECTROCHEMISTRY AND WELDING:

A Practical Treatise on Commercial Chemistry, the Electric Furnace, the Manufacture of Ozone and Nitrogen by High-Tension Discharges, and the Applications of Electric, Gas and Chemical Welding to Manufacturing and Repair Work. Part 1, Applied Electrochemistry, by Charles F. Burgess; Part 2, Welding, by George W. Cravens. Chicago, American Technical Society, 1917. 83 + 132 pp., 186 illus., 8 x 6 in., cloth. $1.50.

A concise account of the principles involved in electrolysis, electrothermics, and electrical discharges in gases, of the apparatus used commercially, and of the products obtained. The section devoted to welding describes the methods of riveting, welding, soldering, and cutting with electricity, gases, and thermit.

CHEMISTRY IN THE SERVICE OF MAN.

By Alexander Findlay. 2d ed. N. Y. and Lond., Longmans, Green and Co., 1917. 15 + 272 pp., 23 illus., 3 por., cloth. $2.00.

An account of some of the more important general principles and theories of chemical science and of their industrial applications, intended for the general reader. A certain amount of new matter has been added to this edition, including a new chapter on Fermentation and Enzyme Action.

CALCULATIONS USED IN CANE SUGAR FACTORIES:

A Practical System of Chemical Control for the Sugar Houses of Louisiana, the Tropics, and Other Cane-Producing Countries. By Irving H. Morse. 2d ed., rewritten. N. Y., John Wiley & Sons, Inc.; Lond., Chapman & Hall, Ltd., 1917. 189 pp., 7 x 4 in., cloth. $2.00.

Intended for technical superintendents and chief chemists of sugar mills.

THE THEORY AND PRACTICE OF SCIENTIFIC MANAGEMENT.

By C. Bertrand Thompson. Bost., N. Y., and Chic., Houghton Mifflin Co. 319 pp., 8 x 5 in., cloth. $1.75.

An attempt to present the facts concerning the subject and to appraise the movement as a whole. Contents: What Scientific Management Is; Frederick Winslow Taylor; Scientific Management in Practice; Economic Aspects of Scientific Management; The Literature of Scientific Management; Bibliography.

SCIENTIFIC OFFICE MANAGEMENT.

By W. H. Leffingwell. N. Y., Chic. and Lond., A. W. Shaw Company (copyright 1917). 253 pp., 32 illus., 3 diagrams, 11 x 9 in., cloth. $10.00.

This is a report on the results obtained by the author during ten years experience in the application to office work of the principles of the Taylor system, including planning, analysis, time and motion study, task and bonus plans, etc. Gives a plan for applying the system.

OFFICE MANUAL.

Including Policy Book and Standard Practice Instructions. N. Y., A. W. Shaw Co. (copyright 1917). 50 pp., 10 x 8 in., cardboard.

A sample office manual supplementing Leffingwell's "Scientific Office Management" and intended to assist managers in preparing a manual for their own establishments.

LIABILITY AND COMPENSATION INSURANCE:

Industrial Accidents and Their Prevention, Employers' Liability, Workmen's Compensation, Insurance of Employers' Liability and Workmen's Compensation. By Ralph H. Blanchard. N. Y. and Lond., D. Appleton & Company, 1917. 12 + 394 pp., 13 illus., 8 x 5 in., cloth. $2.00.

Intended to present the results of the workmen's compensation movement in the United States in terms of legislation and insurance practice, and to explain the industrial accident problem and the development of liability and compensation principles as a background for the comprehension of present problems.

THE ESSENTIALS OF DESCRIPTIVE GEOMETRY.

By F. G. Higbee. 2d ed., rev. N. Y., John Wiley & Sons, Inc.; Lond., Chapman & Hall, Ltd., 1917. 218 pp., 177 illus., 9 x 6 in., cloth. $1.80.

A textbook discussing the subject from the point of view of a draftsman, in which are included only those portions of it which possess industrial utility. In addition to the correction and revision of the text of the first edition, a chapter on tangencies has been added.

THE J. E. ALDRED LECTURES ON ENGINEERING PRACTICE 1916-1917.

The Johns Hopkins University, Department of Engineering. Baltimore, The John Hopkins Press. 254 pp., 82 illus., 9 diagrams, 1 map, 9 x 6 in., pap. (Gift of A. G. Christie.)

Comprises nine lectures delivered before the undergraduate engineering students of the University. The practical phases of engineering problems, rather than the underlying theory, are given prominence. Contents: The Operation of a Hydro-Electric Plant, by A. E. Bauhan; Some Things Engineers Should Know Concerning the Rudiments of Corporate Finance, by Ralph D. Mershon; The Development of Power from the Standpoint of the Boiler Room, by C. F. Hirshfeld; Power and Service in Industrial Plants, by R. J. S. Pigott; Gas Manufacture, Construction, and Operation, by George P. Marrow; Rapid Transit Problems in American Cities, by

George Staples Rice; Some Practical Problems Met with in the Design and Construction of Bridges and Similar Structures, by W. W. Pagon; Experimental Engineering, Particularly the Construction of Testing Stations on Water and Sewerage Problems, by Langdon Pearse; Public Utility Engineering and Finance, by Herbert A. Wagner.

HENDRICKS' COMMERCIAL REGISTER OF THE UNITED STATES, 1917-1918:

For Buyers and Sellers, with Which has been Incorporated "The Assistant Buyer," Especially Devoted to the Interests of the Architectural, Contracting, Electrical, Engineering, Hardware, Iron, Mechanical, Mill, Mining, Quarrying, Railroad, Steel, and Kindred Industries. N. Y., S. E. Hendricks Co., Inc., 1917. 2 227 pp., 10 x 8 in., cloth. $10.00.

Enlarged from the previous edition by more than 400 pages. Includes, as a new section, an alphabetic list of the firms and individuals appearing in the classified Trades Section.

INGENIERIA DE FERROCARRILES:

La Teoria y Practica Fundamental de Ferrocarriles, Desde la Concepcion del Idea Hasta la Terminacion del Trazo. Por Verne LeRoy Havens. N. Y., John Wiley & Sons, Inc.; Lond., Chapman & Hall, Ltd., 1917. 10 + 357 pp., 48 illus., 1 diagram, 7 x 4 in., cloth. $3.50.

A pocketbook for railway surveyors and locating engineers in Latin America, intended to meet the need for such a book in Spanish. Covers the entire work, from the preliminary commercial considerations to the final survey, including the documents and data required to accompany the surveys when submitted to South American Governments.

PRACTICAL ROAD BUILDING.

By Charles E. Foote. Endorsed by the National Highways Association and the American Automobile Association. Philadelphia, David McKay, 1917. 13 + 295 pp., 40 illus., 7 x 5 in., cloth. $1.25.

A manual of information on the construction, maintenance, and advantages of the various types of roads used in America, intended to summarize present-day expert knowledge on the subject in a form intelligible to the general public.

PRACTICAL STRUCTURAL DESIGN:

A Text and Reference Work for Engineers, Architects, Builders, Draftsmen, and Technical Schools; Especially Adapted to the Needs of Self-Tutored Men. By Ernest McCullough. N. Y., U. P. C. Book Company, Inc., 1917. 303 pp., 185 illus., 9 x 6 in., cloth. $2.50.

The subject-matter of this book is based on a series of articles on the "Design of Beams, Girders and Trusses", which appeared in *Building Age* during 1914, but includes much additional material.

WOOD AND OTHER ORGANIC STRUCTURAL MATERIALS.

By Charles Henry Snow. N. Y., McGraw-Hill Book Co., Inc.; Lond., Hill Publishing Co., Ltd., 1917. 18 + 478 pp., 90 illus., 9 x 6 in., cloth. $5.00.

The materials described are woods, paints, and varnishes—with their associated oils, pigments, gums, and resins—glues, creosotes, and india rubber. The author presents the general as well as the physical characteristics of this group of materials, gives in convenient tabular form descriptions of the various woods, discusses the failure of wood because of age, decay, fire, and animal attacks, and the methods of protection from these destructive agencies. Full bibliographic notes are included.

THE BUILDING ESTIMATOR'S REFERENCE BOOK:

A Practical and Thoroughly Reliable Reference Book for Contractors and Estimators Engaged in Estimating the Cost of and Constructing All Classes of Modern Buildings; Giving the Actual Labor Costs and Methods Employed in the Erection of Some of Our Present Day Structures, Together with All Necessary Material Prices and Labor Quantities Entering Into the Cost of All Classes of Buildings. By Frank R. Walker. 2d ed., rev. Chic., Frank R. Walker, 1917. 3535 pp., 7 x 5 in., leather. $5.00. (Gift of the author.)

Presents costs compiled from the actual cost sheets, and chiefly from construction work under the author's supervision.

WATER-SUPPLY ENGINEERING:

The Designing and Constructing of Water-Supply Systems. By A. Prescott Folwell. 3d ed., rewritten. N. Y., John Wiley & Sons, Inc.; Lond., Chapman & Hall, Ltd., 1917. 15 + 484 pp., 121 illus., 9 x 6 in., cloth. $3.50.

In this edition the matter treating of water purification and the quality of water has been entirely rewritten at much greater length; that describing pumping machinery is practically new throughout, and hardly a paragraph of the original text has been used without some change calculated to bring the subject up to date. To avoid making the book too large, the text relating to construction has been condensed to a single chapter, and that dealing with maintenance and hydraulics has been omitted.

OPERATION AND MAINTENANCE OF IRRIGATION SYSTEMS.

By S. T. Harding. N. Y., McGraw-Hill Book Company, Inc.; Lond., Hill Publishing Company, Ltd., 1917. 12 + 271 pp., 27 illus., 9 x 6 in., cloth. $2.50.

A discussion of the policies and methods used in the United States, in which the general principles are given, together with typical examples of their local applications. Contents: General Maintenance; Maintenance of Irrigation Systems; Organization for Operation and Maintenance; Methods of Delivering Irrigation Water; Measurement of Irrigation Water; Rules and Regulations; Payment for Construction and Operation Charges; General Operation; Operation and Maintenance Accounts; Rules and Regulations.

ROLL OF HONOR

A List of Members of the Society who are Serving in the Army or Navy of the United States or Any of its Allies.*

Abbot, Frederic V. Brig-Gen., Corps of Engrs., N. A., Office, Chf. of Engrs., U. S. A., Washington, D. C.

Ackerman, Alexander S. 1st Lieut., E. O. R. C., Care, Depot Q. M., New York City.

Adams, Edward M. Maj., U. S. A., Care, The Adjutant-Gen., U. S. A., Washington, D. C.

Adams, Milton Jewell. Capt., Co. C, 114th Engrs., Camp Beauregard, Alexandria, La.

Alden, Herbert C. 1st Lieut., C. A. C., N. G. U. S., Fort Schuyler, New York City.

Allen, Franklin R. Capt., E. O. R. C., 3d Co., Fort Leavenworth, Kans.

Allen, Ralph B. 1st Lieut., Co. B, 25th Engrs., Am. Exp. Force.

Allen, Walter Hinds. Civ. Engr., U. S. N.; Lt.-Commander; Public Works Officer, Navy Yard, New York City.

Allison, William F. Maj., E. O. R. C., Am. Exp. Force, France.

Altstaetter, F. W. Col., Corps of Engrs., U. S. A., P. O. Box 216, Grand Rapids, Mich.

Anderson, J. H. 1st Lieut., E. O. R. C., 1st Co., Camp American University, Washington, D. C.

Anderson, W. P. Capt., E. O. R. C., 1st Co., Camp American University, Washington, D. C.

Andrews, J. H. M. Maj., 103d Engrs., Camp Hancock, Augusta, Ga.

Ardery, Edward Dahl. Maj., Corps of Engrs., U. S. A., Am. Exp. Force.

Armstrong, Charles Johnstone. Brig.-Gen.; Chf. Engr., Canadian Army Corps, B. E. F., 604 Royal Trust Bldg., Montreal, Que., Canada.

Armstrong, Merwin. Capt., Co. D, 105th Engrs., Camp Sevier, Greenville, S. C.

Arn, William G. Capt.; Adjutant, 1st Bn., 13th U. S. Engrs. (Ry.), Am. Exp. Force, France.

Asplundh, E. T. Capt.; Supply Officer, 103d Engrs., 28th Div., Camp Hancock, Augusta, Ga.

Atterbury, W. W. Brig.-Gen.; Director Gen. of Transportation, Am. Exp. Force.

Austill, Hurieosco. Capt., E. O. R. C., Co. D, 501st Engr. Service Bn., Am. Exp. Force.

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* This list is made up from replies to a circular forwarded to members of the Society, and others, and is probably neither accurate nor complete. It is requested that the attention of the Secretary be called to any omissions or inaccuracies in order that they may be corrected in subsequent lists.

Ayres, Quincy C. 2d Lieut., E. O. R. C., 1022 Vermont Ave., N. W., Washington, D. C.

Bailey, Lewis P. Capt., E. O. R. C., Co. A, 304th Engrs., Camp Meade, Baltimore, Md.

Bailhache, John G. 2d Lieut., E. O. R. C., Vancouver Barracks, Vancouver, Wash.

Baker, H. S. Capt., E. O. R. C.; Const. Q. M., Camp Bowie, Fort Worth, Tex.

Balch, William H. Capt., E. O. R. C., Camp American University, Washington, D. C.

Bandy, Edward L. 1st Lieut., E. O. R. C., Camp American University, Washington, D. C.

Barber, Norman N. 1st Lieut., E. O. R. C., 6th Engrs., Am. Exp. Force, France.

Barclay, A. J. 1st Lieut., E. O. R. C., Fort Leavenworth, Kans.

Barney, Samuel E. Maj., E. O. R. C. (*Unassigned*), 346 Whitney Ave., New Haven, Conn.

Barstow, Eugene Duston. 1st Lieut., E. O. R. C. (*Unassigned*), Cuyahoga Falls, Ohio.

Bartholomew, B. W. 2d Lieut., E. O. R. C., 301st Engrs., Camp Devens, Ayer, Mass.

Battie, H. S. 1st Lieut., E. O. R. C., 1st Co., Fort Leavenworth, Kans.

Bayliss, Paul. 2d Lieut., E. O. R. C., 3d Co., Fort Leavenworth, Kans.

Beach, Lansing H. Col., Corps of Engrs., U. S. A., 412 Custom House, Cincinnati, Ohio.

Beall, Pendleton. Private, Headquarters Co., 165th U. S. I., 83d Brigade, 42d Div.

Beemer, John A. Capt., E. O. R. C., Vancouver Barracks, Vancouver, Wash.

Beerbower, Dumont. 1st Lieut., E. O. R. C., Camp American University, Washington, D. C.

Begg, R. B. H. Capt., E. O. R. C., Am. Exp. Force, France.

Behrman, I. E. 1st Lieut., E. O. R. C., 26th Engrs., Am. Exp. Force, France.

Belzner, Theodore. 1st Lieut., E. O. R. C., 574 West 176th St., New York City.

Benham, W. L. Maj., Q. M. R. C., Camp Funston, Fort Riley, Kans.

Bennison, Ernest William. 1st Lieut., E. O. R. C., Fort Leavenworth, Kans.

Bensel, J. A. Maj., E. O. R. C., Engr. Depot, Norfolk, Va.

Bixby, William H. Brig.-Gen., U. S. A. (*Retired*), 428 Custom House, St. Louis, Mo.

Black, Dudley F. Capt., E. O. R. C., Fort Leavenworth, Kans.

Black, G. G. Capt., E. O. R. C.; Acting Adjutant, 314th Engrs., Camp Funston, Junction City, Kans.

Black, Roger Derby. Lt.-Col., Corps of Engrs., U. S. A., Headquarters, Am. Exp. Force.

Black, R. P. 1st Lieut., E. O. R. C. (*Unassigned*), Sewanee, Tenn.

Black, William M. Brig.-Gen.; Chf. of Engrs., U. S. A., War Dept., Washington, D. C.

Blackman, J. W. B. Lieut., Canadian Ry. Troops, Canadian Expeditionary Force.

Boesch, Clarence E. Capt., 105th Engrs., Camp Sevier, Greenville, S. C.

Boggs, F. C. Lt.-Col., Corps of Engrs., U. S. A.; Col., 315th Engrs., N. A., Camp Travis, San Antonio, Tex.

Boland, Charles J. 1st Lieut., Signal Corps, U. S. A., 228 Eighth St., Troy, N. Y.

Bolin, Harry W. Sergeant, Co. E., 23d Engrs., Camp Meade, Annapolis Junction, Md.

Bond, P. S. Col., 107th Engrs.; Div. Engr., 32d Div., Camp MacArthur, Waco, Tex.

Boorman, Kitchell M. Sapper, Canadian Engrs., Training Depot, St. Johns, Que., Canada.

Booz, Horace Corey. Maj., Staff of Director-Gen. of Rys., Am. Exp. Force, France.

Bott, C. N. 1st Lieut., E. O. R. C., 312th Engrs., Camp Pike, Little Rock, Ark.

Bowlby, Henry L. Capt. and Regimental Adjt., 20th Engrs., Camp American University, Washington, D. C.

Brewster, Henry B. Capt. and Adjt., 303d Engrs., Camp Dix, Wrightstown, N. J.

Bright, Graham B. Capt. and Regimental Adjt., 305th Engrs. (Pioneers), Camp Lee, Petersburg, Va.

Brooking, J. H. 1st Lieut., Co. B, 12th Engrs. (Ry.), Am. Exp. Force, France.

Brown, Alfred T. Capt., E. O. R. C., Camp American University, Washington, D. C.

Brown, Clarence C. 1st Lieut., Co. E, 12th Engrs. (Ry.), Am. Exp. Force, France.

Brown, Earl I. Col., 317th Engrs., Camp Sherman, Chillicothe, Ohio.

Brown, Elliot C. Lt.-Commander, U. S. N. R., Naval Base, Norfolk, Va.

Brown, H. Whittemore. 2d Lieut., E. O. R. C., 301st Engrs., Camp Devens, Ayer, Mass.

Brown, Marshall W. Maj., E. O. R. C., Am. Exp. Force, P. O. 702, France.

Brown, Robert H. Capt., San. Corps, N. A., 21 West 127th St., New York City.

Buck, Richard S. Maj., 11th Engrs. (Ry.), Am. Exp. Force, France.

Buck, Walter Van. Capt., 23d Engrs., Camp Meade, Annapolis Junction, Md.

Buckwalter, Harris D. Capt., E. O. R. C., 1st Co., Camp American University, Washington, D. C.

Bunker, Stephen S. Capt., E. O. R. C., Co. C, 503d Service Bn., Camp Merritt, Tenafly, N. J.

Burdett, O. L. Capt., Co. A, 25th Engrs., Camp Devens, Ayer, Mass.

Burgess, Harry. Lt.-Col., Corps of Engrs., U. S. A., Am. Exp. Force, France.

Burrows, H. G. 1st Lieut., Co. B, 25th Engrs., Camp Devens, Ayer, Mass.

Burton, William. 1st Lieut., E. O. R. C., 734 Fifteenth St., Washington, D. C.

Bushnell, Howard B. Capt., E. O. R. C., Fort Leavenworth, Kans.

Bushnell, Howard E. Capt., O. R. C., Picatinny Arsenal, Dover, N. J.

Byrd, J. H. Capt., E. O. R. C.

Cameron, Harry Frank. Maj., E. O. R. C., 301st Engrs., Camp Devens, Ayer, Mass.

Camp, George Dashiell. 1st Lieut., E. O. R. C., 310 East Elmira St., San Antonio, Tex.

Carey, Matthew L. Capt., Q. M. R. C., Camp Dix, Wrightstown, N. J.

Carey, William N. Capt. and Adjt., 313th Engrs., Camp Dodge, Des Moines, Iowa.

Carroll, James E. Maj., E. O. R. C., 1682 Lincoln Ave., St. Paul, Minn.

Carty, J. J. Col., Signal Corps, U. S. A.

Caton, John H., 3d. Capt., E. O. R. C., Camp American University, Washington, D. C.

Cattell, William A. Maj., E. O. R. C. (*Unassigned*), Foxcroft Bldg., San Francisco, Cal.

Cherry, Alan Gordon. Regimental Sergeant-Maj., 301st Engrs., Camp Devens, Ayer, Mass.

Chevalier, Willard T. Capt., Co. D, 11th Engrs. (Ry.), Am. Exp. Force, France.

Chittenden, Albert Frederick. 1st Lieut., Co. F, 18th Engrs. (Ry.), Am. Exp. Force, France.

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MEMBERSHIP

(From October 5th to November 8th, 1917)

ADDITIONS

MEMBERS		Date of Membership.
ALLEN, JAMES. Highway Commr., State of Washington, Olympia, Wash		Oct. 9, 1917
BARGLEBAUGH, CHARLES ERWIN. With Lang & Witchell, 703 First National Bank Bldg., El Paso, Tex		Oct. 9, 1917
BARROWS, GEORGE ELLSWORTH. Cons. Engr. (Ellsworth, Barrows & Reeves), 50 West Eagle St., Buffalo, N. Y	Assoc. M. M.	May 2, 1911 Oct. 9, 1917
BATTLE, JOHN BEALLE. Asst., U. S. Engr. Corps, Tuscaloosa, Ala		Oct. 9, 1917
COOKSEY, ROBERT MAVIN. Highways Engr., City Hall, Baltimore, Md	Assoc. M. M.	July 2, 1913 Oct. 9, 1917
CRAGIN, CHARLES CALHOUN. Capt., Co. C, 18th Engrs. (Ry.), Am. Exp. Force, France		Sept. 11, 1917
GRIFFIN, AUGUSTUS. Chf. Engr. and Supt., South San Joaquin Irrig. Dist., Manteca, Cal	Jun. Assoc. M. M.	Oct. 30, 1906 May 28, 1912 Oct. 9, 1917
HAMMEL, VICTOR FRANK. Engr. of Designs, Elec. Bond & Share Co., 71 Broadway, New York City	Assoc. M. M.	Oct. 31, 1911 Oct. 9, 1917
HARZA, LEROY FRANCIS. Chf. Engr., James O. Heyworth, 421 Cedar St., Sault Ste. Marie, Mich		Oct. 9, 1917
HENDERSON, HERBERT. Constr. Engr., Gulf Refining Co., Port Arthur, Tex		Oct. 9, 1917
JESSEN, NILS HERMAN BAASHUUS. Care, Chile Exploration Co., 120 Broadway, New York City		Oct. 9, 1917
MULLEN, CHARLES WARD. Engr. Member, State Public Utilities Comm., Bangor, Me		Sept. 11, 1917
NUMATA, HISANORI. Chf. Engr., Civ. and Archt. Dept., Imperial Govt. Steel Works, Yawata City, Japan		May 15, 1917
PIEPMEIER, BION HARMAN. Maintenance Engr., State Highway Dept., Springfield, Ill	Assoc. M. M.	May 6, 1914 Oct. 9, 1917
PIERCE, PAUL LEON. Maj., Ordnance Dept., U. S. R., 65 Park Ave., New York City	Jun. Assoc. M. M.	Jan. 7, 1908 Oct. 3, 1911 Oct. 9, 1917
REEVE, LEROY NORMAN. Structural Engr., Am. International Shipbuilding Co., Broad and Cherry Sts., Philadelphia, Pa	Assoc. M. M.	Sept. 3, 1913 Oct. 9, 1917
RICE, JOHN MARIE THOMAS. With Morris Knowles, 2541 Oliver Bldg. (Res., 5435 Black St.), Pittsburgh, Pa	Jun. Assoc. M. M.	April 12, 1907 Dec. 6, 1910 June 12, 1917

MEMBERS (*Continued*)

		Date of Membership.
TINSLEY, ROBERT BRUCE. Big Stone Gap, Va.	Assoc. M. M.	Feb. 6, 1912 Oct. 9, 1917
TROTT, DAVID CROOKER. Supt. of Constr., U. S. Public Bldgs., Kalispell, Mont.	Assoc. M. M.	Sept. 6, 1910 Sept. 11, 1917

ASSOCIATE MEMBERS

ANDREEN, GUSTAF BIRGER. Structural Engr., Bldg. Dept., Public Service Ry. of New Jersey, 12 Eighteenth St., Elmhurst, N. Y.		Oct. 9, 1917
ASHTON, RAYMOND. (Cattell, Howard & Ashton), 68 Post St., San Francisco, Cal.		June 11, 1917
BANTA, RUSSELL VINCENT. Structural Engr., 26 Cortlandt St., New York City	Jun. Assoc. M.	May 31, 1910 Oct. 9, 1917
BARSTOW, EUGENE DUSTON. City Engr., 69 North St., Cuyahoga Falls, Ohio		Oct. 9, 1917
BESTOR, ODIN BALTIMORE. Asst. Field Engr., Interstate Commerce Comm., Care, Valuation Dept., Delaware & Hudson Co., Albany, N. Y.		Oct. 9, 1917
BOORMAN, KITCHELL MONCKTON. Sapper, Canadian Engrs. Training Depot, Saint Johns, Que., Canada		Oct. 9, 1917
BOWNE, SIDNEY BREESE. Secy. and Treas., W. E. Sexton Co., Inc., Mineola, N. Y.	Jun. Assoc. M.	April 18, 1916 Oct. 9, 1917
BROWN, HORACE TROWBRIDGE. Asst. Res. Engr., California Highway Comm., 26 Third St., San Francisco, Cal.		Oct. 9, 1917
CARLSON, PAUL NELS. With Grant Smith-Porter-Guthrie Co., Multnomah Hotel, Portland, Ore.		Oct. 9, 1917
CHAMBERLIN, CHARLES GROVER. Engr.-in-Chg. of Appraisal of Kansas Gas & Elec. Co., for Paine, McClellan & Campion, 237 South Main St., Wichita, Kans.		Oct. 9, 1917
CRAWFORD, HUGH WILLIAM. Capt., Co. A, 110th Engrs., 35th Div., Camp Doniphan, Fort Sill, Okla.		Oct. 9, 1917
CULPEPER, HORACE. Office and Field Man, Gulf Pipe Line Co., Houston, Tex.		Oct. 9, 1917
CUTLER, DANIEL BOYDEN. Asst. Engr., St. L. S. W. Ry., 4399 Forest Park, St. Louis, Mo.		May 15, 1917
DE WITT, BRINTON BROWN. Civ. Engr., Central Fe, Salamanca, Santa Clara, Cuba		Sept. 11, 1917
DRESSER, GEORGE LUCAS. Bridge Engr., Jennison Wright Co.; Contr. Engr., Standard Eng. Co. of Toledo, Ohio, 2632 Whitehall Bldg., New York City		Oct. 9, 1917
EDDY, HAROLD MANSFIELD. With Pickands, Mather & Co. of Cleveland, 1136 Walnut Ave., N. E., Canton, Ohio		Oct. 9, 1917
FARNER, GEORGE KEITHLEY. Asst. Engr., C., M. & St. P. Ry., 1009 Majestic Bldg., Milwaukee, Wis.		Oct. 9, 1917

ASSOCIATE MEMBERS (*Continued*)

		Date of Membership.
FERGUSON, HARRY FOSTER. Lieut., 311th Engrs., Camp Grant, Rockport, Ill.	Jun. Assoc. M.	June 23, 1916 Oct. 9, 1917
FRANK, LESLIE CARL. San. Engr., U. S. Public Health Service, Hygienic Laboratory, 25th and E Sts., Washington, D. C.	Jun. Assoc. M.	Dec. 3, 1913 Sept. 11, 1917
FULLER, MYRON ELMER. 7131 Cresheim Rd., Philadelphia, Pa.		Sept. 11, 1917
GRIFFIN, GEORGE APPLETON. Asst. Engr., Board of Water Supply, Room 2200, Municipal Bldg., New York City.		April 17, 1917
HADLEY, HOMER MORE. Designer, V. D. Simons, 1212 Standard Bank Bldg., Vancouver, B. C., Canada	Jun. Assoc. M.	April 5, 1910 Oct. 9, 1917
HAMMOND, LEWIS MERRICK. Structural Steel Work Draftsman, Bureau of Yards and Docks, Navy Dept. (Res., 234 V St., N. E.), Washington, D. C.	Jun. Assoc. M.	Mar. 2, 1915 Oct. 9, 1917
HARBERT, GUY MORLEY. County Road Engr., Harrison County, Clarksburg, W. Va.		Oct. 9, 1917
HEALY, FREDERIC GEORGE. Asst. Constr. Engr., Remington Arms & Union Metallic Cartridge Co., Inc., 586 Arctic St., Bridgeport, Conn.		Oct. 9, 1917
HOUSEHOLDER, FRANK JOHNSON. Industrial Engr., 405 Law Bldg., Baltimore, Md.		Oct. 9, 1917
JAKOBSEN, BERNHARD FAABORG. Designing Engr., F. G. Baum & Co. (Inc.), 1902 Hobart Bldg., San Francisco, Cal.		Oct. 9, 1917
JONES, WILLIAM DONELSON. Constr. Engr., Los Angeles Shipbuilding & Dry Dock Co., 783 Eleventh St., San Pedro, Cal.		Oct. 9, 1917
KAISER, BENEDICT JOSEPH. Designing and Superv. Engr., Prack & Perrine, 6961 Bennett St., Pittsburgh, Pa.		Oct. 9, 1917
KORNFELD, HARRY. With Hugh L. Cooper & Co., 101 Park Ave, New York City (Res., 90 East 18th St., Brooklyn, N. Y.)	Jun. Assoc. M.	Sept. 3, 1912 Oct. 9, 1917
LLOYD, HENRY. Chf. Engr., Carey Act Dept., State Land Office, Capitol Bldg., Cheyenne, Wyo.		June 11, 1917
LOHR, WILLIAM SHANNON. Engr., John Wickersham, 638 North Pine St., Lancaster, Pa.		Oct. 9, 1917
McCLURE, HUNTER. Junior Civ. Engr., Div. of Valuation, Interstate Commerce Comm., 731 Wells Fargo Bldg., San Francisco, Cal.	Jun. Assoc. M.	Jan. 4, 1910 Oct. 9, 1917
McCOOL, HARRY ENGLAND. Designer, Duluth & Iron Range R. R., 244 West Faribault St., Duluth, Minn.		Oct. 9, 1917

ASSOCIATE MEMBERS (*Continued*)

		Date of Membership.
McKinnell, Frederic Berkby. Engr., A., T. & S. F. Ry., Care, Chf. Engr., A., T. & S. F. Ry., Topeka, Kans...		Sept. 11, 1917
McMillan, Franklin R. Asst. Prof., Structural Eng., Univ. of Minnesota, 524 Eighth Ave., S. E., Minneapolis, Minn....		Sept. 11, 1917
Massei, Caesar. Maj., Va. C. A.; Fort Commander, Fisherman's Island, Fort Monroe, Va....	Jun. Assoc. M.	Jan. 2, 1912 Sept. 11, 1917
Mead, William Henry. Chf. Engr., Producers Oil Co., Box 1735, Houston, Tex....		Oct. 9, 1917
Mickey, Clark Edwin. Asst. Prof. of Applied Mech., Univ. of Nebraska; Cons. and Testing Engr., Lincoln, Nebr....	Assoc. Assoc. M.	Jan. 6, 1915 Oct. 9, 1917
Miller, Melville Sampson. Asst. Engr., Public Service Comm., 972a St. Johns Pl., Brooklyn, N. Y....		Oct. 9, 1917
Moore, Stanley Wallace. Estimator, Fred T. Ley & Co., Inc., 47 Cedar St., Springfield, Mass....	Jun. Assoc. M.	July 1, 1909 Sept. 11, 1917
Paine, Allen Thatcher. Asst. Engr., New York State Highway Comm., 1 Third St., Oneonta, N. Y....		Sept. 11, 1917
Perkins, Alva Harold. Capt., E. O. R. C., 17th Engrs. (Ry.), Am. Exp. Forces, France....		Oct. 9, 1917
Pickett, Frank Hurd. With Am. Graphophone Co., 1112 Howard Ave., Bridgeport, Conn....		Oct. 9, 1917
Riblet, Harry Gaillard. Engr. with Metz & Roth, 144 East 8th St., Erie, Pa....	Jun. Assoc. M.	Jan. 31, 1911 Oct. 9, 1917
Rockwell, Harlow Loveridge. Supervisor of Subway, New York Telephone Co. (Res., 207 Woodward Ave.), Buffalo, N. Y....		Oct. 9, 1917
Rose, Alston Orange. Asst. Engr., Morris Knowles, 1200 B. F. Jones Bldg., Pittsburgh, Pa....	Jun. Assoc. M.	July 2, 1913 Oct. 9, 1917
Salle, George Viall. Supt., The Foundation Co., New York City, 665 West Front St., Cincinnati, Ohio....		Oct. 9, 1917
Sammelman, Charles William Sylverius. Engr., Board of Public Service, 325 City Hall, St. Louis, Mo....		Oct. 9, 1917
Schmucker, Beale Melancthon. 1st Lieut., 104th Engrs., U. S. R., Camp McClellan, Ala....		Oct. 9, 1917
Segura, Valeriano. Dist. Engr., Bohol Prov., Bureau of Public Works, Box 23, Tagbilaran, Bohol, Philippine Islands....	Jun. Assoc. M.	Oct. 1, 1913 May 15, 1917
Shaffer, Ivan Oscar. Concrete Engr., American Can Co., 120 Broadway, Room 1361, New York City....		Oct. 9, 1917
Sharp, Herbert Moore. 2301 Maplewood Ave., Toledo, Ohio....		Sept. 11, 1917

ASSOCIATE MEMBERS (*Continued*)

		Date of Membership.
SHARP, HOMER J. With Union Oil Co. of California, 1311 Union Oil Bldg., Los Angeles, Cal.	Jun. Assoc. M.	April 4, 1911 Sept. 11, 1917
SHIMER, HARRY WOODWORTH. Bureau of Eng., City Hall, San Francisco, Cal.		Sept. 11, 1917
SHIPLEY, ROBERT EARLE. Drainage Engr., Texarkana Pipe Works, Texarkana, Ark.-Tex.		Oct. 9, 1917
SMITH, MAXWELL WAIDE. Capt., 308th Engrs., U. S. R., Camp Sherman, Chillicothe, Ohio		April 17, 1917
STEVENS, SAMUEL SPAULDING. Asst. Field Engr., Div. of Valuation, Interstate Commerce Comm., East. Dist., Washington, D. C.		Oct. 9, 1917
STOFFLET, HERBERT MILLER. Asst. Engr., C., M. & St. P. Ry., 612 East 41st St., Chicago, Ill.		Oct. 9, 1917
STUART, EDWARD. Care, Engineer's Society, Harrisburg, Pa.		Oct. 9, 1917
SWINTON, ROY STANLEY. Instr., Eng. Mech., Univ. of Michigan, 1126 Washtenaw Ave., Ann Arbor, Mich.	Jun. Assoc. M.	May 2, 1911 Sept. 11, 1917
TORPEN, BERNHARDT E. Care, Cerro de Pasco Min. Co., La Oroya, Peru		April 17, 1917
VAN AMBURGH, THOMAS ALBERT. 225 Insurance Bldg., Oklahoma, Okla.		May 15, 1917
VAN DEN BROEK, JOHN ABRAHAM. Instr. in Eng. Mechanics, Univ. of Michigan, 819 South State St., Ann Arbor, Mich.		Oct. 9, 1917
VERNER, JULIUS. Chf. Draftsman, Gen. Eng. Dept., Standard Oil Co. (N. J.), Box 206, Linden, N. J.		Oct. 9, 1917
VODGES, JOSEPH JOHNSON. Engr., Brann & Stuart Co., 310 Commercial Trust Bldg., Philadelphia, Pa.		Oct. 9, 1917
WALTON, HARRISON BILLINGSLEY. Capt., E. O. R. C., 312th Engrs., Camp Pike, Little Rock, Ark.		Oct. 9, 1917
WAND, ANTHONY WILLIAM. Prof. of Civ. Eng., Univ. of New Mexico, Albuquerque, N. Mex.		Sept. 11, 1917
WARD, LYMAN WISE. Capt., Coast Artillery Corps, Officers Reserve, U. S. A., Fort Worden, Wash.		April 17, 1917
WATERMAN, EARLE LYTTON. Associate Prof., San. Eng., The Pennsylvania State Coll., 312 South Burrowes St., State College, Pa.		Sept. 11, 1917
WENDT, WYLIE BRODBECK. Asst. Prof., Applied Mechanics, Kansas State Agri. Coll.; Engr. of Tests, State Highway Comm., Manhattan, Kans.		Oct. 9, 1917
WIEDEMAN, HERMAN FREDERICK. Engr., Solomon-Norcross Co., 1625 Candler Bldg., Atlanta, Ga.		Oct. 9, 1917
WINN, HARRY STRONG. Senior Civ. Engr., Interstate Commerce Comm., Interstate Bldg., Kansas City, Mo.	Jun. Assoc. M.	July 9, 1912 Oct. 9, 1917

JUNIORS	Date of Membership.
BICKEL, GEORGE ROBERT. With Bickel Asphalt Co., 1315 Cherokee Rd., Louisville, Ky.	Oct. 9, 1917
BREYMANN, JOHN BERNARD, JR. 2052 Robinwood Ave., Toledo, Ohio	May 15, 1917
FRITZ, WALTER. 502 East 5th St., Brooklyn, N. Y.	Oct. 9, 1917
JACOBS, NATHAN BERND. Asst. Engr., Morris Knowles, 1200 Jones Bldg., Pittsburgh, Pa.	Sept. 11, 1917
KEITH, GERALD MARCY. 2d Lieut., E. O. R. C., 301st Engrs., Camp Devens, Ayer, Mass.	Sept. 11, 1917
LEFEVER, KENNETH WINANS. Instrumentman and Draftsman, Ford & MacCrea, 326 Gazette Bldg., Little Rock, Ark.	Sept. 11, 1917
PARKER, JAMES. Asst. to Gen. Supt., Roxana Petroleum Co. of Oklahoma, Box 422, Tulsa, Okla.	Oct. 9, 1917
RADER, JAMES WILSON. County Surv., Greenbrier County, Lewisburg, W. Va.	April 17, 1917
RENSHAW, ALFRED. Capt., 302d Engrs., Camp Upton, N. Y.	Oct. 9, 1917
REPKO, GEORGE ANTON. Queens Rd., Queens, N. Y.	June 11, 1917
RITOW, HERMAN. Co. B, 302d Engrs., Camp Upton, N. Y.	Oct. 9, 1917
SEXTON, GEORGE FRANCIS. Care, Const. Quartermaster, Camp Jackson, Columbia, S. C.	May 15, 1917
STRACHAN, NORMAN FRASER. Asst. Engr., Kansas State Board of Health, 1339 Ohio St., Lawrence, Kans.	Oct. 9, 1917
TRUESDELL, STEPHEN RIGGS. Asst. Engr., Office of Div. Engr., M. C. R. R., 265 West Main St., Jackson, Mich.	Oct. 9, 1917

CHANGES OF ADDRESS

MEMBERS

ABBOT, FREDERIC VAUGHN. Brig.-Gen., Office, Chf. of Engrs., U. S. A., Washington, D. C.

ANDREWS, HORACE. Cocoanut Grove, Fla.

BACON, JAMES HAYWARD. 1507 St. Johns Ave., Jacksonville, Fla.

BARNES, MORTIMER GRANT. Cons. Engr., 36 State St., Albany, N. Y.

BERRY, JOHN BENNINGTON. Cons. Engr. (Berry & Roberts), 109 North Kenilworth Ave., Oak Park, Ill.

BROWN, EARL IVAN. Col., 317th Engrs., N. A., Camp Sherman, Chillicothe, Ohio.

BROWNELL, ERNEST HENRY. Civ. Engr., U. S. N., Care, Miss Brownell, 8 Newberry St., Boston, Mass.

BURKY, CHARLES ROGY. Co. 1, R. O. T. C., Presidio, San Francisco, Cal.

CARPENTER, EDWARD EMERY. Executive Engr., Sanderson & Porter, Raymond, Wash.

CASE, MONTGOMERY BARCOCK. Res. Engr., Metropolis Bridge, Metropolis, Ill.

MEMBERS (*Continued*)

CASLER, MELVIN DAVID. 143 Stevens Ave., Mount Vernon, N. Y.

CHILD, STEPHEN. Landscape Archt. and Cons. Engr., Care, Pierce & Barnes Co., 7 Water St., Boston, Mass.

COE, CLARENCE STANLEY. Capt., 17th Engrs., (Ry.), Am. Exp. Force, France.

COLLINS, EMMETT FILMORE. Valuation Engr., St. L. & S. F. R. R., 631 Frisco Bldg., St. Louis, Mo.

COURTNEY, REGINALD SYDNEY. Care, British War Mission, 1735 Equitable Bldg., New York City.

CUNNINGHAM, JOHN GEORGE LAWRENCE. Hotel Angus, St. Paul, Minn.

DARROW, WILTON JOSEPH. 344 West End Ave., New York City.

DATER, PHILIP HERRICK. Cons. Engr., 1000 Chamber of Commerce Bldg. (Res., 1345 Wistaria Ave.), Portland, Ore.

DAVIS, CHANDLER. Capt., 6th U. S. Engrs., Washington Barracks, Washington, D. C.

DE LA MATER, STEPHEN TRUESDELL. Cons. Engr., 1040 McCormick Bldg., Chicago, Ill.

DENT, ELLIOTT JOHNSTONE. Col., 26th Engrs., Camp Dix, N. J.

EARLY, PERCY WALKER. Care, Mason & Hanger Co., Lake Charles, La.

ENDICOTT, MORDECAI THOMAS. (*Past-President*). Civ. Engr., U. S. N.; Rear-Admiral (*Retired*), 1865 Wyoming Ave., N. W., Washington, D. C.

EVANS, JOSEPH DEAN. 30 Church St., Suite 501-E, New York City.

FICKES, CLARK ROBINSON. 110 North Clinton St., Iowa City, Iowa.

FISHER, JANON. Maj., E. O. R. C., Eccleston, Md.

FRENCH, FRANK CHAUNCEY. Mgr., Consolidated Min. & Milling Co., Lake City, Colo.

GARDINER, JOHN PEDEN. 222 California Ave., Santa Monica, Cal.

GAYOL, ROBERTO. Cons. Engr., Apartado Postal 766, City of Mexico, Mexico.

GERBER, WINFRED DEAN. Civ. and San Engr., Chamber of Commerce, Chicago, Ill.

GIBSON, NORMAN ROTHWELL. 190 University Ave., Toronto, Ont., Canada.

GOODALE, LOOMIS FARRINGTON. 215 McGregor Boulevard, Fort Myers, Fla.

HAGEMAN, HARRY ANDREW. Mech. Engr., Fore River Shipbuilding Corporation, Quincy, Mass.

HAINES, HENRY STEVENS. 1154 Worthington St., Springfield, Mass.

HAINS, PETER CONOVER. Maj. Gen., U. S. A. (*Retired*); Dist. Engr., Box 283, Norfolk, Va.

HAMLIN, HOMER. Cons. Engr. and Geologist, 1103 Central Bldg., Los Angeles, Cal.

HARRISON, WILLIAM BURR. Maj., E. O. R. C., War Dept., Office of Chf. of Engrs., Washington, D. C.

HEWINS, GEORGE SANFORD. 640 Middle St., Portsmouth, N. H.

HITTELL, JOHN BENJAMIN. Cons. Engr., Lincoln, Ill.

MEMBERS (*Continued*)

HOOD, WILLIAM. Chf. Engr., S. P. Co., 65 Market St., Room 1057, San Francisco, Cal.

HOXIE, RICHARD LEVERIDGE. Brig.-Gen., U. S. A. (*Retired*), 1632 K St., N. W., Washington, D. C.

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HUGHES, FRANCIS DEY. Chf. Engr., Contr. Dept., Illinois Steel Bridge Co., 416 Title Guaranty Bldg., St. Louis, Mo.

JACOBS, JULIUS LILIEN. Southern Mgr., James Stewart & Co., Inc., Box 556, Norfolk, Va.

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JONAH, FRANK GILBERT. Maj., 12th Engrs. (Ry.), Am. Exp. Force, France.

KEMP, JOHN EDWARD. E. O. T. C., Fort Leavenworth, Kans.

KNIGHT, RICHARD WARREN. Sales Dept., Austin Co., Cleveland, Ohio.

KUTZ, CHARLES WILLAUER. Lt.-Col., Corps of Engrs., U. S. A., Am. Exp. Forces, France.

LEWIS, EVERETT WILSON. 9 Armory St., Springfield, Mass.

LOEWENSOHN, DAVID. Engr. and Contr., 12415 Saywell Ave., Cleveland, Ohio.

LUND, ALFRED MAJENDIE. 2126 Arch St., Little Rock, Ark.

MACREDIE, JOHN ROBERT CLARKE. 1220 Third Ave., N. W., Moose Jaw, Saskatchewan, Canada.

McCORMICK, HERBERT GRANVILLE. Gaffney, S. C.

McGONIGLE, CHARLES JOSEPH. Pres., Western Structural Steel & Tank Co., 13th and Pettygrove Sts., Portland, Ore.

MANTON, ARTHUR WOODROFFE. Care, J. W. Woods, Esq., Director of Purchases, British War Mission, 1735 Equitable Bldg., New York City.

MARTIN, DANIEL HOWARD. Chf. Engr., James H. Corbett, 839 Chestnut St., Indiana, Pa.

MINNISS, GEORGE STEWART. Maj., 74th N. Y. Infantry, Spartanburg, S. C.

MINOR, CYRUS EDWARD. Engr. of Constr., Rawley Mine, Box 668, Salida, Colo.

MOULTON, HERBERT GEORGE. Cons. Engr., Care, War Industries Board, Council of National Defense, 18th and D Sts., N. W., Room 818, Washington, D. C.

MUKASA, SEITARO. Chf. Engr., Civ. Eng. Section, Kawasaki Dock Yard Co., Kobe, Japan.

NASH, FRANKLYN DANA. Care, 12th Engrs., Am. Exp. Forces, France.

NEVILLE, COLONE WILL JACKSON. Lt.-Commander, U. S. N., N. V., 927 Am. National Insurance Bldg., Galveston, Tex.

PARKER, PHILIP À MORLEY. 25 Victoria St., London, S. W., England.

PECK, MYRON HALL. Capt., 305th Engrs., Care, Adjutant-Gen., U. S. A., Camp Lee, Va.

PHILLIPS, WILLIAM HALE. 717 Columbian Ave., Oak Park, Ill.

MEMBERS (*Continued*)

PIERCE, PAUL LEON. Maj., Ordnance Dept., U. S. R., 65 Park Ave., New York City.

RICE, WALTER PERCIVAL. Cons. Engr., The Walter P. Rice Eng. Co., 420 Erie Bldg., Cleveland, Ohio.

ROUSSEAU, HARRY HARWOOD. Rear-Admiral, U. S. N.; Member, Comm. on Navy Yards and Naval Stations, Navy Dept., 1619 Massachusetts Ave., Washington, D. C.

SAYERS, EDWARD LAWRENCE. Care, Anniston Steel Co., Anniston, Ala.

SCHMIDT, MAX EBERHARDT. Pres. and Chf. Engr., Continuous Transit Securities Co., 20 Broad St., Room 706, New York City.

SHAW, PERCY AUGUSTUS. 430 Walnut St., Manchester, N. H.

SHIPLEY, HENRY FRANKLIN. Prin. Asst. Engr., City Eng. Dept., In Chg. of Highway Div. (Res., 289 Erkenbrecker Ave.), Cincinnati, Ohio.

SLOCUM, HARRY SPENCER. Res. Engr., Vielé, Blackwell & Buck, Glen Lyn, Va.

SMEAD, RAPHAEL CHART. U. S. Asst. Engr., Dist. Engr. Office, Galveston, Tex.

SMITH, WILSON FITCH. Res. Engr., U. S. Shipping Board, Emergency Fleet Corporation, 506 Finance Bldg., Philadelphia, Pa.

STEVENS, WILLIAM WENTWORTH. Technology Club, 17 Gramercy Park, New York City.

SWEETSER, CHARLES HERBERT. Dist. Engr., Office of Public Roads and Rural Eng., U. S. Dept. of Agriculture, Mills Bldg., San Francisco, Cal.

THOMPSON, MILTON THEODORE. 35 Woodside Ave., Ridgewood, N. J.

TODD, FRANK HERBERT. Maj., Q. M. Corps; Officer in Chg. of Utilities, Camp Travis, San Antonio, Tex.

TROST, ADOLPHUS GUSTAVUS. Archt. (Trost & Trost), 815 Mills Bldg., El Paso, Tex.

WADDELL, ROBERT WILLIAM. 3205 Highland Ave., Kansas City, Mo.

WALKER, ELTON DAVID. Capt., Co. A, 15th U. S. Engrs., Am. Exp. Forces, France.

WARE, JOHN. 1st Lieut., 101st U. S. Engrs., Am. Exp. Forces, France.

WAUGH, WILLIAM HAMMOND. Capt., E. O. R. C.; Engr. Officer, Alaska Road Comm.; Senior Highway Engr., Office of Public Roads and Rural Eng., Juneau, Alaska.

WAUTERS, CARLOS. Prof., National Univ. of Buenos Aires; Civ. and Const. Engr., Belzrano 456, Buenos Aires, Argentine Republic.

WHISTLER, JOHN T. Engr., Irrig. and Drainage, Federal Farm Loan Bureau, 432 Federal Bldg., Denver, Colo.

WILD, HERBERT JOSEPH. Capt., E. O. R. C., 1316 New Hampshire Ave., N. E., Washington, D. C.

WILSON, JOHN. 511 West 32d St., Austin, Tex.

WING, CHARLES BENJAMIN. Maj., 23d Engrs., Camp Meade, Admiral, Md.

WOODARD, WILKIE. 1114 West 51st St., Los Angeles, Cal.

ASSOCIATE MEMBERS

ADAMS, MILTON JEWELL. Capt., Co. C, 114th Engrs., Camp Beauregard, Alexandria, La.

AMADON, FREDERICK WEBBER. Computer, Interstate Commerce Comm., Div. of Valuation, 5112 Thirteenth St., N. W., Washington, D. C.

ARMSTRONG, GEORGE SIMPSON. Care, Miller, Franklin & Co., Union National Bank Bldg., Cleveland, Ohio.

ATKINSON, ARTHUR GARRATT. 2115 Third Ave., Sacramento, Cal.

AYERS, MURRAY CHASE. Chf. of Party on Road Surveys, California Highway Comm., 2266 Sierra Madre St., Pasadena, Cal.

BECKJORD, JESSEE GARNET. Care, California Highway Comm., Los Angeles, Cal.

BEHRMAN, ISADORE ELLIS. 1st Lieut., E. O. R. C., 26th Engrs., Am. Exp. Force, France.

BLAAUW, GEERT. 311 Sixth St., Jackson, Mich.

BLECH, EDWARD SORREL. Care, Carlota Mines, Cienfuegos, Cuba.

BORCHERS, PERRY ELMER. Supt. of Constr., Indian Service at Large, 210 East Taylor St., Phœnix, Ariz.

BRETT, LAWRENCE. Pres. and Gen. Mgr., Brett Eng. & Contr. Co., 327 Monticello Arcade, Norfolk, Va.

BRIGHT, DUDLEY SEYMOUR. Dist. Sales Mgr., Concrete Steel Co., Room 524, New Guardian Bldg., Cleveland, Ohio.

BUCK, ROSS JUDSON. Asst. Engr., Indianapolis Water Co., 2204 North Alabama St., Indianapolis, Ind.

BURROWES, PAUL DE NYSE. Point Pleasant, N. J.

CARPENTER, J. C. U. S. Highway Engr., Box 176, West Raleigh, N. C.

CARR, DEAN ORRICE. 5030 Evans Ave., Chicago, Ill.

CHRISTOPHERS, REGINALD GILLON. No. 60286, 2d Lieut., 34th Reinforcements, New Zealand Exp. Force, Care, G. P. O., Wellington, New Zealand.

CLACK, JAMES MORGAN. Contr. Engr., Baldwin, Fla.

CLAPP, SIDNEY KINGMAN. Asst. Engr., Board of Water Supply, Grand Gorge, N. Y.

COLLINS, JERRY. Supt., The Dock Contractor Co. of Hoboken, 81 Berwyn St., Orange, N. J.

COLTMAN, ROBERT, JR. Asst. Engr., Public Service Comm., First Dist., 64 Elliott Ave., Yonkers, N. Y.

CONWAY, NORMAN BUTLER. Capt., E. O. R. C., 6th Co., E. O. T. S., Fort Leavenworth, Kans.

CORNELL, CHARLES BROWN. 1496 South High St., Columbus, Ohio.

COYNE, HARRY LEWIS. Asst. Engr., Public Service Comm., First Dist., 111 Hanover Ave., Elmhurst, N. Y.

CRANE, ALBERT ELI. 1st Lieut., Co. A, 6th Engrs., Washington Barracks, Washington, D. C.

CROOKES, HAROLD JOSEPH. Care, Chester Shipbuilding Co., Ltd., Chester, Pa.

ASSOCIATE MEMBERS (*Continued*)

CURREY, JOHN WAGGONER. 1st Lieut., E. O. R. C., Engr. Field Depot, Laredo, Tex.

DEAN, DANIEL ABRAM. Contr. Engr., United Iron Works Co., Box 386, Iola, Kans.

DIXON, DE FOREST HALSTED. Second Vice-Pres., Turner Constr. Co., 244 Madison Ave., New York City (Res., 71 Gates Ave., Montclair, N. J.)

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RESIGNATIONS

ASSOCIATE MEMBERS	Date of Resignation.
DOWNER, THOMAS BENSON	Oct. 9, 1917
FOGG, PERCIVAL MORRIS	Oct. 9, 1917
SKINNER, FREDERICK GARDINER	Oct. 9, 1917

DEATHS

ANDREWS, DANIEL MARSHALL. Elected Member, March 2d, 1892; died June, 1917.

BACOT, WILLIAM SINCLAIR. Elected Member, October 1st, 1890; died October 31st, 1917.

CHITTENDEN, HIRAM MARTIN. Elected Member, February 7th, 1900; died October 9th, 1917.

COLBY, ELMER ELLSWORTH. Elected Member, January 7th, 1913; died September 27th, 1917.

GOULD, HARRY MADERA. Elected Member, November 30th, 1909; died September 30th, 1917.

MARR, WILLIAM WALTER. Elected Member, February 2d, 1909; died October 3d, 1917.

SWANKER, JOHN EDWARD. Elected Member, May 4th, 1904; died October 20th, 1917.

WACHTEL, LOUIS. Elected Junior, March 2d, 1909; Associate Member, September 2d, 1914; died October 10th, 1917.

Total Membership of the Society, November 8th, 1917,

8 534.

MONTHLY LIST OF RECENT ENGINEERING ARTICLES OF INTEREST

(August 2d to September 1st, 1917)

NOTE.—*This list is published for the purpose of placing before the members of this Society, the titles of current engineering articles, which can be referred to in any available engineering library, or can be procured by addressing the publication directly, the address and price being given wherever possible.*

LIST OF PUBLICATIONS

In the subjoined list of articles, references are given by the number prefixed to each journal in this list:

(2) *Proceedings*, Engrs. Club of Phila., Philadelphia, Pa.
(3) *Journal*, Franklin Inst., Philadelphia, Pa., 50c.
(4) *Journal*, Western Soc. of Engrs., Chicago, Ill., 50c.
(5) *Transactions*, Can. Soc. C. E., Montreal, Que., Canada.
(6) *Journal*, Am. Inst. Architects, Washington, D. C., 50c.
(7) *Gesundheits Ingenieur*, München, Germany.
(8) *Stevens Indicator*, Hoboken, N. J., 50c.
(9) *Industrial Management*, New York City, 25c.
(11) *Engineering* (London), W. H. Wiley, 432 Fourth Ave., New York City, 25c.
(12) *The Engineer* (London), International News Co., New York City, 35c.
(13) *Engineering News-Record*, New York City, 15c.
(15) *Railway Age Gazette*, New York City, 15c.
(16) *Engineering and Mining Journal*, New York City, 15c.
(17) *Electric Railway Journal*, New York City, 10c.
(18) *Railway Review*, Chicago, Ill., 15c.
(19) *Scientific American Supplement*, New York City, 10c.
(20) *Iron Age*, New York City, 20c.
(21) *Railway Engineer*, London, England, 1s. 2d.
(22) *Iron and Coal Trades Review*, London, England, 6d.
(23) *Railway Gazette*, London, England, 6d.
(24) *American Gas Engineering Journal*, New York City, 10c.
(25) *Railway Mechanical Engineer*, New York City, 20c.
(26) *Electrical Review*, London, England, 4d.
(27) *Electrical World*, New York City, 10c.
(28) *Journal*, New England Water-Works Assoc., Boston, Mass., $1.
(29) *Journal*, Royal Society of Arts, London, England, 6d.
(30) *Annales des Travaux Publics de Belgique*, Brussels, Belgium, 4 fr.
(31) *Annales* de l'Assoc. des Ing. Sortis des Ecoles Spéciales de Gand, Brussels, Belgium, 4 fr.
(32) *Mémoires et Compte Rendu des Travaux*, Soc. Ing. Civ. de France, Paris, France.
(33) *Le Génie Civil*, Paris, France, 1 fr.
(34) *Portefeuille Economiques des Machines*, Paris, France.
(35) *Nouvelles Annales de la Construction*, Paris, France.
(36) *Cornell Civil Engineer*, Ithaca, N. Y.
(37) *Revue de Mécanique*, Paris, France.
(38) *Revue Générale des Chemins de Fer et des Tramways*, Paris, France.
(39) *Technisches Gemeindeblatt*, Berlin, Germany, 0, 70m.
(40) *Zentralblatt der Bauverwaltung*, Berlin, Germany, 60 pfg.
(41) *Electrotechnische Zeitschrift*, Berlin, Germany.
(42) *Proceedings*, Am. Inst. Elec. Engrs., New York City, $1.
(43) *Annales des Ponts et Chaussées*, Paris, France.
(44) *Journal*, Military Service Institution, Governors Island, New York Harbor, 50c.
(45) *Coal Age*, New York City, 10c.
(46) *Scientific American*, New York City, 15c.
(47) *Mechanical Engineer*, Manchester, England, 3d.
(48) *Zeitschrift*, Verein Deutscher Ingenieure, Berlin, Germany, 1, 60m.
(49) *Zeitschrift für Bauwesen*, Berlin, Germany.
(50) *Stahl und Eisen*, Düsseldorf, Germany.
(51) *Deutsche Bauzeitung*, Berlin, Germany.
(52) *Rigasche Industrie-Zeitung*, Riga, Russia, 25 kop.
(53) *Zeitschrift*, Oesterreichischer Ingenieur und Architekten Vereines, Vienna, Austria, 70h.
(54) *Transactions*, Am. Soc. C. E., New York City, $12.
(55) *Transactions*, Am. Soc. M. E., New York City, $10.
(56) *Transactions*, Am. Inst. Min. Engrs., New York City, $6.

(57) *Colliery Guardian*, London, England, 5d.
(58) *Proceedings*, Engrs.' Soc. W. Pa., 2511 Oliver Bldg., Pittsburgh, Pa., 50c.
(59) *Proceedings*, American Water-Works Assoc., Troy, N. Y.
(60) *Municipal Engineering*, Indianapolis, Ind., 25c.
(61) *Proceedings*, Western Railway Club, 225 Dearborn St., Chicago, Ill., 25c.
(62) *American Drop Forger*, Thaw Bldg., Pittsburgh, Pa., 10c.
(63) *Minutes of Proceedings*, Inst. C. E., London, England.
(64) *Power*, New York City, 5c.
(65) *Official Proceedings*, New York Railroad Club, Brooklyn, N. Y., 15c.
(66) *Gas Journal*, London, England, 6d.
(67) *Cement and Engineering News*, Chicago, Ill., 25c.
(69) *Der Eisenbau*, Leipzig, Germany.
(71) *Journal*, Iron and Steel Inst., London, England.
(71*a*) *Carnegie Scholarship Memoirs*, Iron and Steel Inst., London, England.
(72) *American Machinist*, New York City, 15c.
(73) *Electrician*, London, England, 18c.
(74) *Transactions*, Inst. of Min. and Metal., London, England.
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(76) *Brick*, Chicago, Ill., 20c.
(77) *Journal*, Inst. Elec. Engrs., London, England, 5s.
(78) *Beton und Eisen*, Vienna, Austria, 1, 50m.
(79) *Forscherarbeiten*, Vienna, Austria.
(80) *Tonindustrie Zeitung*, Berlin, Germany.
(81) *Zeitschrift für Architektur und Ingenieurwesen*, Wiesbaden, Germany.
(83) *Gas Age*, New York City, 15c.
(84) *Le Ciment*, Paris, France.
(85) *Proceedings*, Am. Ry. Eng. Assoc., Chicago, Ill.
(86) *Engineering-Contracting*, Chicago, Ill., 10c.
(87) *Railway Engineering and Maintenance of Way*, Chicago, Ill., 10c.
(88) *Bulletin* of the International Ry. Congress Assoc., Brussels, Belgium.
(89) *Proceedings*, Am. Soc. for Testing Materials, Philadelphia, Pa., $5.
(90) *Transactions*, Inst. of Naval Archts., London, England.
(91) *Transactions*, Soc. Naval Archts. and Marine Engrs., New York City.
(92) *Bulletin*, Soc. d'Encouragement pour l'Industrie Nationale, Paris, France.
(93) *Revue de Métallurgie*, Paris, France, 4 fr. 50.
(95) *International Marine Engineering*, New York City, 20c.
(96) *Canadian Engineer*, Toronto, Ont., Canada, 10c.
(98) *Journal*, Engrs. Soc. Pa., Harrisburg, Pa., 30c.
(99) *Proceedings*, Am. Soc. of Municipal Improvements, New York City, $2.
(100) *Professional Memoirs*, Corps of Engrs., U. S. A., Washington, D. C., 50c.
(101) *Metal Worker*, New York City, 10c.
(102) *Organ für die Fortschritte des Eisenbahnwesens*, Wiesbaden, Germany.
(103) *Mining and Scientific Press*, San Francisco, Cal., 10c.
(104) *The Surveyor and Municipal and County Engineer*, London, England, 6d.
(105) *Metallurgical and Chemical Engineering*, New York City, 25c.
(106) *Transactions*, Inst. of Min. Engrs., London, England, 6s.
(107) *Schweizerische Bauzeitung*, Zürich, Switzerland.
(108) *Southern Machinery*, Atlanta, Ga. 10c.
(109) *Journal*, Boston Soc. C. E., Boston, Mass., 50c.
(110) *Journal*, Am. Concrete Inst., Philadelphia, Pa., 50c.
(111) *Journal of Electricity, Power and Gas*, San Francisco, Cal., 25c.
(112) *Internationale Zeitschrift für Wasser-Versorgung*, Leipzig, Germany.
(113) *Proceedings*, Am. Wood Preservers' Assoc., Baltimore, Md.
(114) *Journal*, Institution of Municipal and County Engineers, London, England, 1s. 6d.
(115) *Journal*, Engrs.' Club of St. Louis, St. Louis, Mo., 35c.
(116) *Blast Furnace and Steel Plant*, Pittsburgh, Pa., 15c.

LIST OF ARTICLES

Bridges.

Pavement Problems on New York Bridges.* **(60)** Aug.
Rebuilding Susquehanna River Bridge, Harrisburg. **(67)** Aug.
Cantilever Concrete Bridge has Suspended Link.* **(13)** Aug. 2.
Bridge Work on the Chalco-Yutan Cut-Off. **(15)** Aug. 3.
Kentucky Road Department has Standard Bridges. **(13)** Aug. 9.
Freezing Ground Acts Like Hydraulic Jack.* H. J. Gilkey. **(13)** Aug. 23.
Building a Bridge Under Adverse Conditions in San Domingo.* E. S. Needham. **(13)** Aug. 23.
Construction of the Thames River Bridge, N. Y., N. H. & H. R. R.* **(18)** Aug. 25.
Points in Steel-Bridge Maintenance.* **(13)** Aug. 30.
Die Aufstellung der Hell-Gate-Brücke über den East-River in New York.* O. H. Ammann. **(107)** Aug. 4.

* Illustrated.

Electrical.

A Successful Electric Truck Delivery Service.* F. D. Weber. (111) Aug. 1.
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* Illustrated.

PAPERS AND DISCUSSIONS

NOVEMBER, 1917

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INSTITUTED 1852

PAPERS AND DISCUSSIONS

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Vol. XLIII. NOVEMBER, 1917. No. 9.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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PAPERS AND DISCUSSIONS

CONSTRUCTION PROBLEMS OF THE MANHATTAN-BRONX, AND LEXINGTON AVENUE SUBWAY JUNCTION AND QUEENSBOROUGH TUNNEL CONNECTIONS

By George Perrine, M. Am. Soc. C. E.

To be Presented December 19th, 1917.

Synopsis.

The object of this paper is to describe the principal construction problems that confronted the contractor in making the connection between the present subway in Park Avenue and the new Lexington Avenue line in the vicinity of 42d Street, Borough of Manhattan, New York City, and also in the extension of the Queensborough Tunnel.

The greater part of this contract is made up of special construction. As the standard construction in the New York subways has often been described, it will not be dealt with in this paper.

General Description.

On September 11th, 1914, the Public Service Commission for the First District of New York invited proposals for the construction of Section No. 1 of Route 43, a part of the Seventh Avenue Rapid Transit Railroad, and Section No. 1 of Route 26, a part of the Steinway Tunnel, now called the Queensborough Tunnel. The Commis-

Note.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and when finally closed, the papers, with discussion in full, will be published in *Transactions*.

sion included in this proposal a spur to the existing Manhattan-Bronx Rapid Transit Railroad.

The limits of the routes covered by this paper are as follows: Section No. 1 of Route 43 (Fig. 1) extends from a point about 50 ft. south of the south side of 38th Street to the north side of 42d Street. It also includes the spur-track connecting with the present subway at Vanderbilt Avenue, and the ramp from the Queensborough Elevator Shaft to the Grand Central Station. Section No. 1 of Route 26 is in Forty-second Street, below the Diagonal Station, and extends from Vanderbilt Avenue to a point about 100 ft. east of the east building line of Lexington Avenue.

The contract was executed on December 4th, 1914, and the time allowed for the completion of the work was 28 months.

TABLE 1.—PRINCIPAL QUANTITIES.

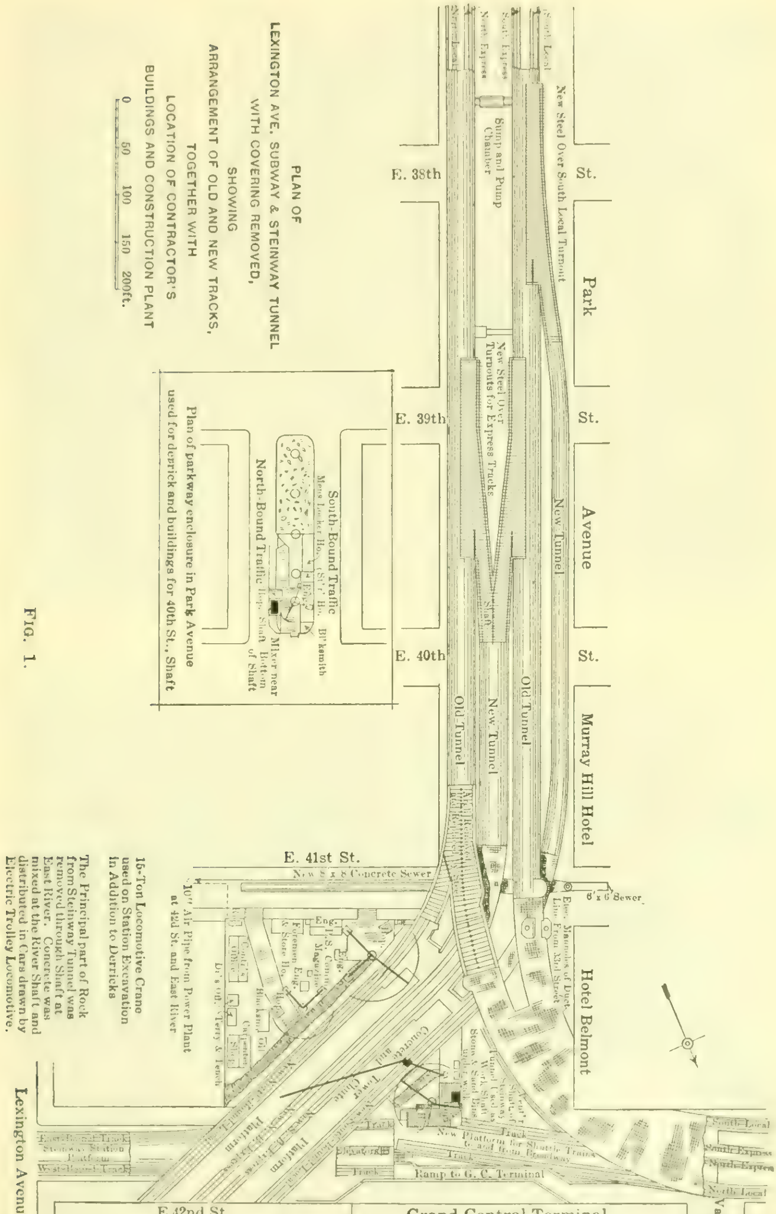
	Section 1, Route 43.	Section 1, Route 26.
Earth excavation, in cubic yards.....	65 000	300
Rock excavation, " " "	45 000	130
Tunnel excavation, " " "	25 500	24 900
Steel, in tons..........................	4 725	309
Concrete, in cubic yards..............	26 000	11 850

The general plan of construction called for a sub-surface railroad having four tracks in the case of Route 43, and sub-surface railroads having two tracks in the cases of Route 26 and the spur.

The work under the contract included the care and support of buildings, sewers, pipes, railroads, with their rolling stock, and other surface, sub-surface, and overhead structures; the maintenance of traffic; the safety and protection of passengers and other persons; the restoration of pavements and other surfaces; and the removal and reconstruction of portions of the Manhattan-Bronx Rapid Transit Railroad and portions of the existing Queensborough Tunnel, in order to provide a connection with the new work.

The contract also provided that the adjacent portions of the Queensborough Tunnel be put into operation before the completion of the work under contract, and that the work be conducted in such a manner as not to interfere with or interrupt the safe and continuous operation of the trains, and avoid injury to passengers or other persons.

Shafts for the Disposal of Excavated Materials.—The contract required that the work on Section 1, Route 43, be prosecuted from



PLAN OF
LEXINGTON AVE. SUBWAY & STEINWAY TUNNEL
WITH COVERING REMOVED,
SHOWING
ARRANGEMENT OF OLD AND NEW TRACKS,
TOGETHER WITH
LOCATION OF CONTRACTOR'S
BUILDINGS AND CONSTRUCTION PLANT
0 50 100 150 200ft.
Plan of parkway enclosure in Park Avenue
used for derrick and buildings for 40th St., Shaft
South-Bound Traffic
North-Bound Traffic
Mixer near Bottom of Shaft
E. 38th
St.
E. 39th
St.
E. 40th
St.
E. 41st St.
E. 42nd St.
Park
Avenue
Murray Hill Hotel
Hotel Belmont
Grand Central Terminal
Lexington Avenue
Vanderbilt Ave.
North Local
North Express
South Express
South Local
Sump and Pump Chamber
New Steel Over South Local Turnout
New Steel Over Turnouts for Express Trucks
New Tunnel
Old Tunnel
Shaft
New 8 x 8 Concrete Sewer
6'x 6' Sewer
10" Air Pipe from Power Plant at 42d St. and East River
15-Ton Locomotive Crane used on Station Excavation in Addition to Derricks
The Principal part of Rock from Steinway Tunnel was removed through Shaft at East River. Concrete was mixed at the River Shaft and distributed in Cars drawn by Electric Trolley Locomotive.
Ramp to G. C. Terminal

FIG. 1.

within the limits of the property acquired by the City, bounded by the east line of Park Avenue, the north line of East 41st Street, and the south line of East 42d Street. This was formerly the site of the Grand Union Hotel, the buildings on which had been torn down and removed by other contractors.

All material was taken into or removed from the property through a driveway across the north sidewalk of East 41st Street.

Excavated material from the Queensborough Tunnel level was taken up through the shaft east of First Avenue after it had been transmitted through the north tube—which the contract provided could be used for construction purposes—and also up through the ventilating shaft on Park Avenue, near the southeast corner of Park Avenue and East 42d Street.

The contractor was permitted to have an additional shaft in Park Avenue adjacent to the east side of the parkway at East 40th Street. This shaft will be back-filled after the completion of the work.

Excavation.—The excavation for the Diagonal Station crossing the lot east of Park Avenue, between 41st and 42d Streets, was done entirely as open cut. The material, both earth and rock, was removed by derricks and locomotive crane. The portion of the work in Park Avenue, between 42d and 40th Streets, and in 42d Street, was done by the usual cut-and-cover method. The south-bound local track, running south from the diagonal station, was completed by tunneling. The Queensborough Tunnel, which was originally a two-track road, each track in a separate tunnel, was widened by the slice method, and thus the two single-track tunnels were changed into a single-span arch, forming the new Grand Central Station.

Sewers.—The sewers to the west of Park Avenue were led into a large chamber built just west of the new south-bound local tunnel.

Many changes were made necessary in the sewer system on account of the construction of these sections. The most important part of the sewer reconstruction was that running from Park Avenue and 41st Street. It consisted of three 42-in. cast-iron pipes, under all the subways, and an 8 by 8-ft. concrete sewer to the East River. The contractor built this section to a point 200 ft. east of Third Avenue.

Duct Lines.—The duct banks of the present subway, from 33d to 42d Street, had been laid between the tracks in the north-bound

FIG. 2.—OPEN CUT, LOOKING TOWARD 41ST STREET, SHOWING DRIFTS FOR LOCAL AND EXPRESS TUNNELS.

FIG. 3.—GREAT ARCH COMPLETED. STATION PLATFORM ADJOINING EAST SIDE OF ELEVATOR SHAFT.

and south-bound tunnels, and had to be removed on account of the turnouts from the old to the new tunnels at 38th and 39th Streets. The contract required that a new duct line of 72 conduits be built in Park Avenue from 33d to 41st Street, and that connections be made with the present lines in splicing chambers built at 33d and 41st Streets. At these streets, duct lines were built, crossing the roof of the present subway, so that connections could be made at either side with the old conduits. At Fifth Avenue and 42d Street another connection was made, crossing the present subway, on account of the change in the tracks in 42d Street in front of the Grand Central Terminal.

The Steinway or Queensborough Tunnel.

What is now known as the Queensborough Tunnel is the final development of that which was originally called the Steinway Tunnel. It was begun under a franchise granted to William Steinway in 1890. Owing to a serious accident, the work of construction had been abandoned.

In 1905 the New York and Long Island Railroad Company, a subsidiary of the Interborough Rapid Transit Company, purchased the Steinway franchise, and, under a contract with the Degnon Contracting Company, drove two single-track tunnels under the East River. These extended west to Park Avenue, ending in a loop (Fig. 4). On September 26th, 1907, the tunnels were completed, and a trolley car with an overhead feed made its initial trip from Jackson Avenue Station, in Long Island City, to what is now known as the Grand Central Station of the Queensborough Tunnel, just east of Lexington Avenue.

During construction the tunnel was usually spoken of as the Belmont Tunnel, August Belmont being at the time President of the Interborough Transit Company.

Owing to litigation over the alleged expiration of the franchise, the running of the car was discontinued, and the tunnel remained unused until October, 1907, when the car was removed from the tunnel.

In 1913 the tunnels were purchased by the City and incorporated as a section of the Dual Subway System. The work of reconstruction from an overhead trolley system to one operated by the third rail was commenced. The tunnels were opened to public traffic on June 22d,

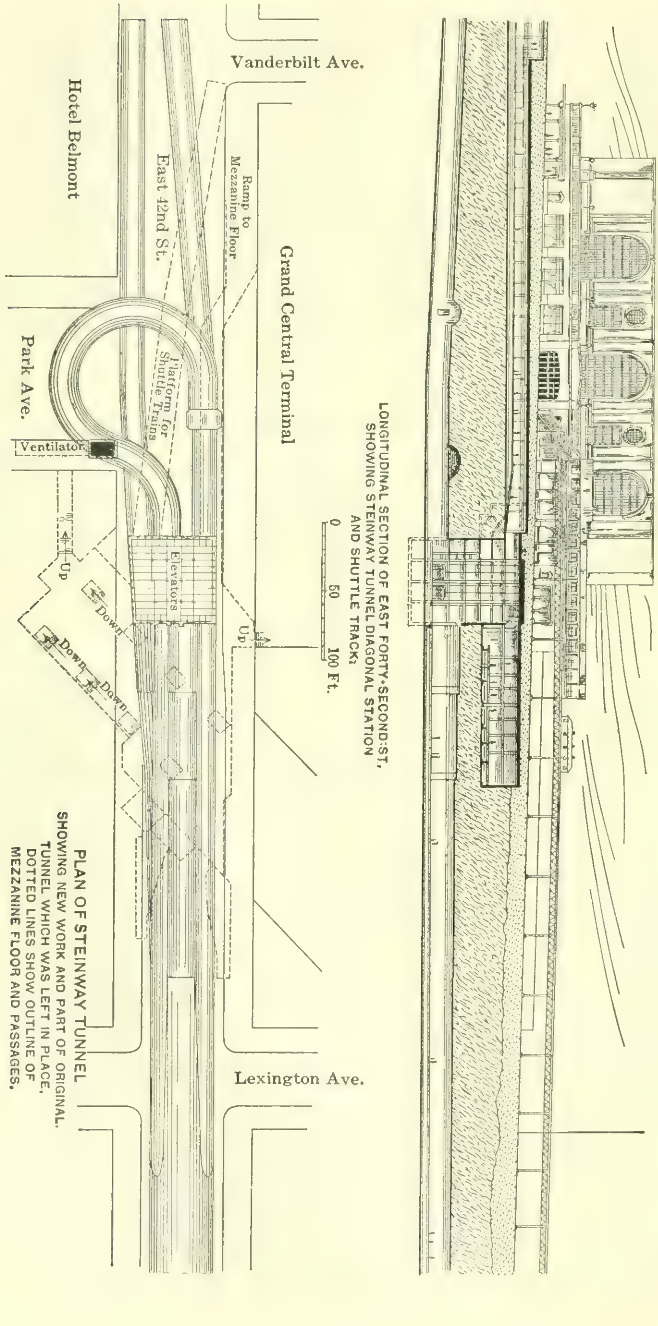
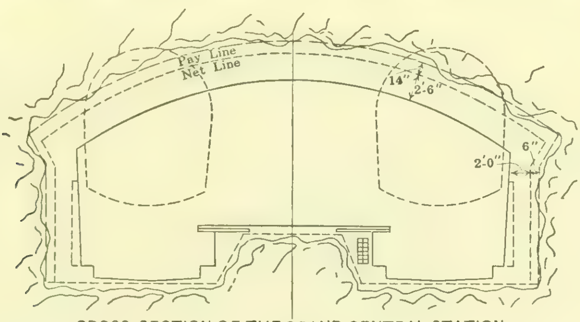

Vanderbilt Ave.
Hotel Belmont
East 42nd St.
Ramp to Mezzanine Floor
Grand Central Terminal
Park Ave.
Platform for Shuttle Trains
Ventilator
Elevators
Up
Down
Down
Down
Up
0
50
100 Ft.
Lexington Ave.
LONGITUDINAL SECTION OF EAST FORTY-SECOND ST.
SHOWING STEINWAY TUNNEL DIAGONAL STATION
AND SHUTTLE TRACK.
PLAN OF STEINWAY TUNNEL
SHOWING NEW WORK AND PART OF ORIGINAL
TUNNEL WHICH WAS LEFT IN PLACE.
DOTTED LINES SHOW OUTLINE OF
MEZZANINE FLOOR AND PASSAGES.

Fig. 4.

1915, between the Jackson Avenue Station and the so-called Grand Central Station in East 42d Street, near Lexington Avenue. The tunnel is now in operation to the Bridge Plaza, in Long Island City, where it connects with the Interborough Rapid Transit Company's system extending to Corona. When the present subway line, now running westward from the Grand Central Terminal, is diverted to the Lexington Avenue route, it is intended to extend the Queensborough Tunnel westward from the Grand Central Terminal to Times Square, by an ascending grade, until it reaches the level of and joins the two south-bound tracks of the present 42d Street Subway, between Fifth and Sixth Avenues.

The original Steinway Tunnel, where the change was made to conform to the new design, was a single-track railway tunnel running east and west in East 42d Street, with a circular return loop in Park Avenue.



CROSS-SECTION OF THE GRAND CENTRAL STATION AS BUILT

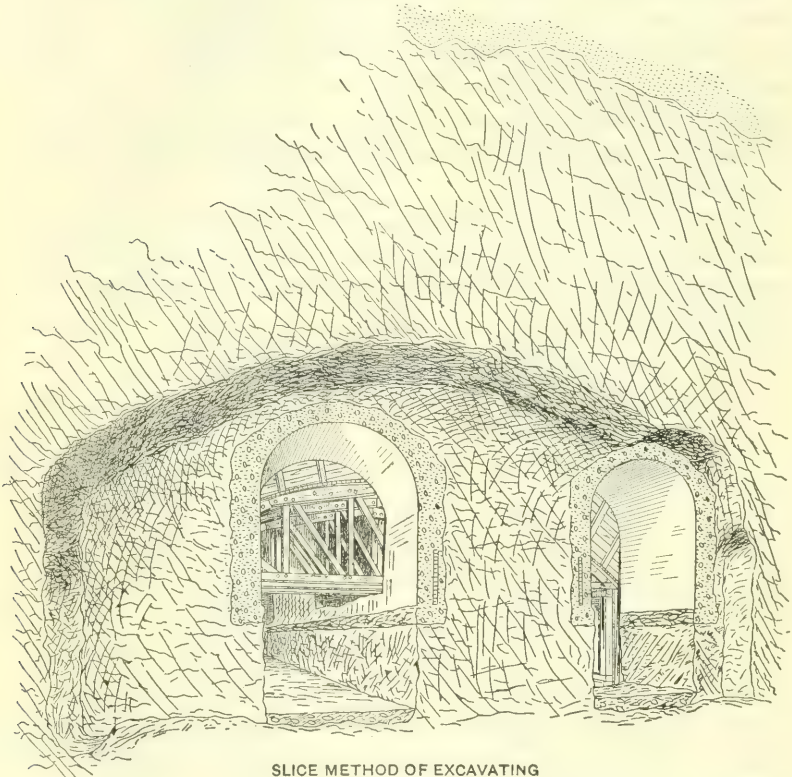
FIG. 5.

The two single tunnels were about 65 ft. to base of rail below the street surface, and were 28 ft. from center to center, with 12 ft. of rock between the tunnel walls.

Method of Reconstructing the Queensborough Tunnel to Extend the Grand Central Station to the Elevator Shaft.—Fig. 5 shows the relative positions of the old and new tunnel arches, and also shows the tunnel as reconstructed.

The first operation of reconstruction was to excavate the inverts of the two single-track tunnels and the rock down to the new subgrade throughout the length of the new station. After that was done, cross-cuts were made at intervals, connecting the two old tunnels; they were 12 ft. wide and in height they extended from a little below

the springing line of the arch to the neat line of the crown. The cross-cut having been completed for the full extent of the new arch, the concrete side-walls were built up to the springing line of the arch. The next operation was to erect the timber centers and bulkheads above, ready to receive the concrete. After the concrete had been



SLICE METHOD OF EXCAVATING
FOR THE
RECONSTRUCTION OF THE STEINWAY TUNNEL

1.–Slices 15 ft. wide were excavated through the walls of the old tunnels, and then the arch form was erected.
2.–After concreting the ring the material adjoining it is excavated and the arch is built.

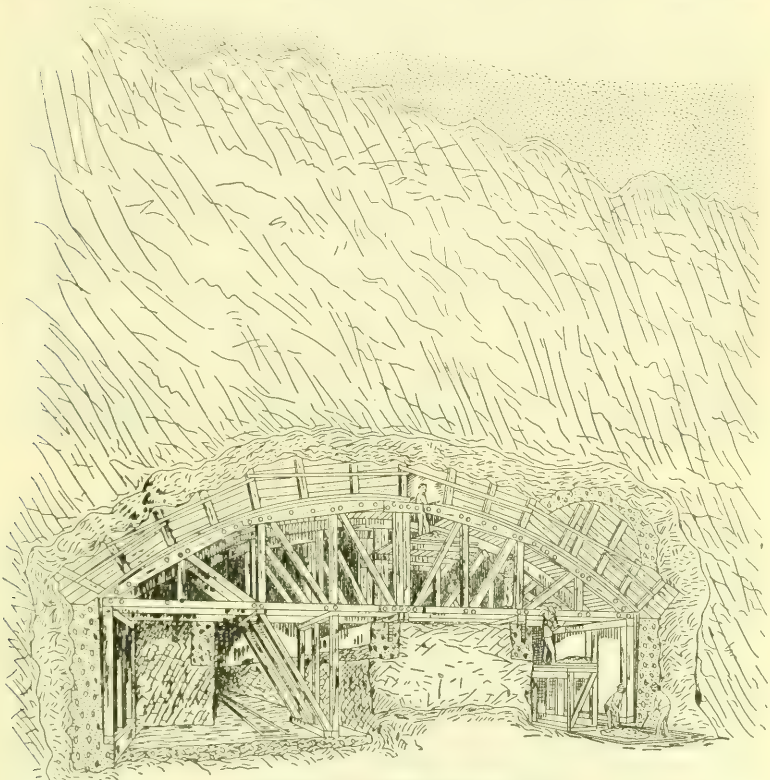
FIG. 6.

placed, and while it was setting, the drilling for the advance cut was completed for another setting of the arch centers.

The contract provided for water-proofing above the large arch, but, after the excavation had advanced considerably, it was decided to omit it. The arch was all grouted and water-proofed inside by the hydro-

lithic cement plaster process. The only timber used during the progress of the excavation for the large arch was that in the concrete arch center.

The concrete for the great arch was placed by hand. It was mixed at Shaft 2, at the East River, and hauled from the mixer to the work by an electric trolley through the north tube. The concrete was dumped



METHOD OF PLACING CONCRETE ARCH
Span of arch, 45 ft. 6 in. to 55 ft. 1¼ in.
Concrete was delivered in iron side-dump cars.
FIG. 7.

from the cars on platforms on the tunnel floor below the arch centers, and cast up into place by shovelers. Two men worked above the lagging, placing and tamping the concrete of the arch. A gang of fourteen men, working 24 hours, in 8-hour shifts, could fill one arch ring, comprising 100 cu. yd. of concrete.

Figs. 6 and 7 show the method of procedure in excavation and in concrete placing.

Queensborough Tunnel Elevator Shaft.—Another feature of the Queensborough Tunnel construction that was as important as the great arch was the elevator shaft. This shaft is about 63 ft. square to the neat line of the excavation, and is 71 ft. in depth to the sub-grade of the track.

The first operation in preparing to sink the shaft was to deck the street in the usual manner. When the shaft was started, the cut for the diagonal station was down to the sub-grade crossing the lot to the south of 42d Street. A cut was excavated from the lot, parallel to the west side of the shaft and near it, over to the north curb of 42d Street.

Before excavating this cut it was necessary to reinforce the brick sewer in 42d Street where it crossed the cut. This reinforcement consisted of steel rods in concrete envelopes. The excavated material from the cross-cut to the shaft and from the upper portion of the first section of the shaft was removed from the lot by the derrick, previously erected to handle the excavated material from the 42d Street ventilating shaft. Until it was possible to break through the roof of the Queensborough Tunnel, the muck was hoisted in buckets, suspended from the girders of the street decking, and loaded into cars. Four drifts were dug from the 42d Street lot, northward, to the curb in front of the Grand Central Terminal. These drifts were proportioned to receive lattice girders to support the timber decking of the street. These girders were from 40 to 44 ft. long and from 42 to 52 in. deep, and were procured from the Second and Third Avenue Elevated Railway during the three-track reconstruction. The north portion of the shaft was excavated first and the lattice girders, after being placed, were supported on posts to rock at the edge of the shaft. One end of each girder was near the north curb of 42d Street, the other end was on the center line of the elevator pit.

After the decking girders were in place, the excavation for the full north half of the shaft was begun. Several pipe lines, including a 30-in. water pipe, crossed the north half of the shaft excavation; also, while the sinking of the north portion of the shaft was in progress, the sewer crossing the south portion was by-passed in a 40-in. steel pipe.

When the mezzanine floor was completed, the decking girders for spanning the south half of the shaft were in place, ready to take the load of the street and the sidewalk from the temporary posting.

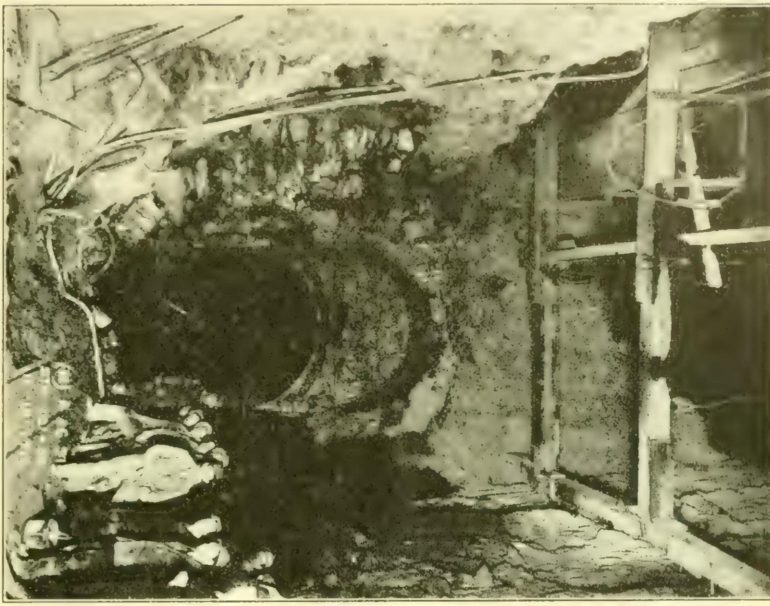


FIG. 8.—EXCAVATION FOR ELEVATOR SHAFT, LOOKING WEST.

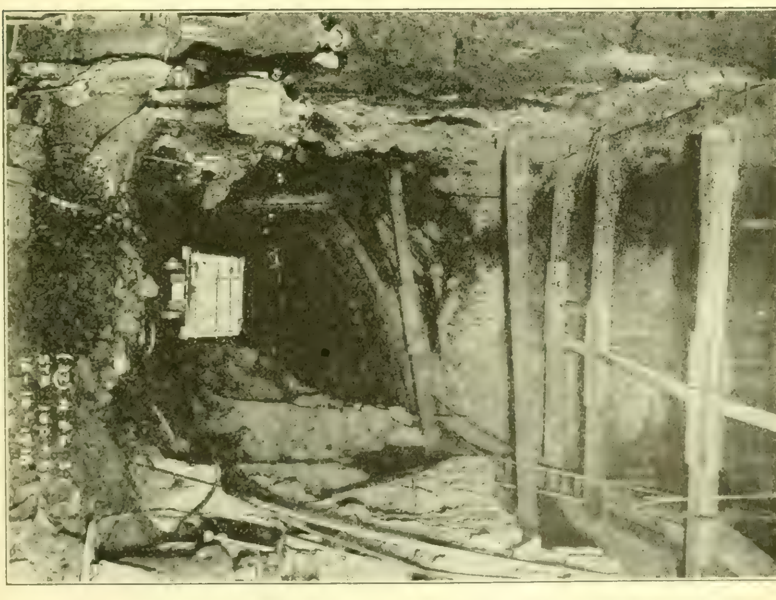


FIG. 9.—NEW STATION ARCH OF QUEENSBORO TUNNEL, LOOKING EAST.

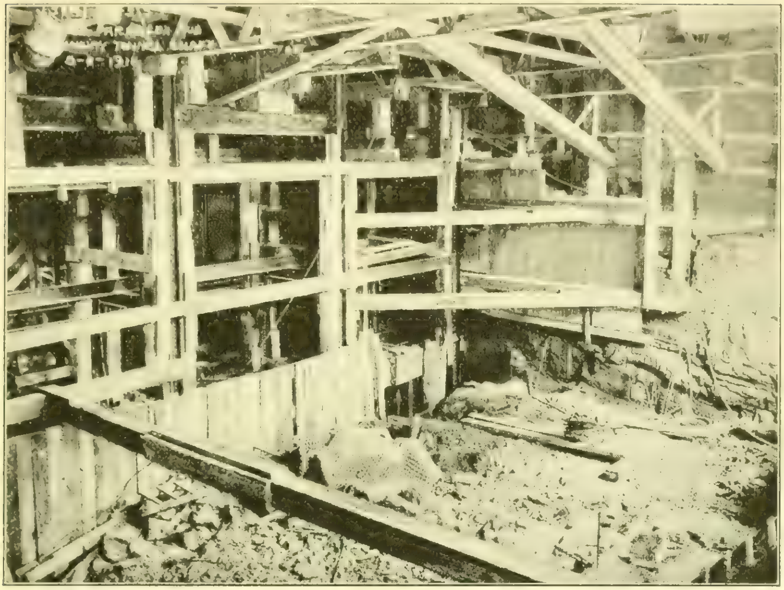


FIG. 10.—EXCAVATION FOR SOUTH SIDE OF ELEVATOR SHAFT COMPLETED TO GRADE OF ENGINE-ROOM FLOOR.

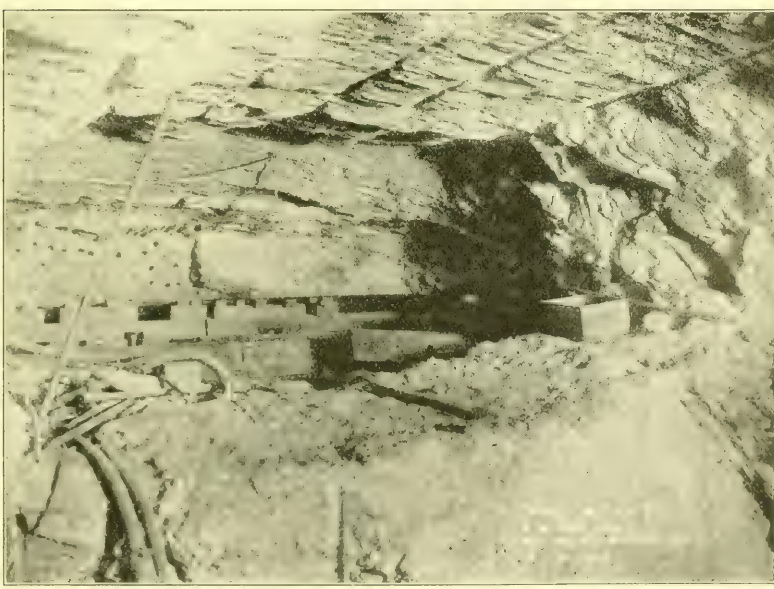


FIG. 11.—REAR OF SIDE DRIFT, SHOWING OLD ARCH DRILLED. BLOCKING IS SHOWN UNDER ARCH RESTING ON SHIELD.

Two 52-in. girders were placed at the same time as the decking girders for supporting those columns of the Elevated Railroad that came within the limit of the south part of the shaft excavation. The north ends of the decking and underpinning girders were all supported on the steel of the mezzanine floor, and this had to be reinforced in order to carry the loads properly distributed.

In connection with the Queensborough Tunnel there is a ventilating shaft near the southeast corner of 42d Street and Park Avenue. This shaft was joined with the south tube of the double tunnel section which runs west from the elevator shaft. These two single-track tunnels were included in the contract as far as Vanderbilt Avenue. The south tunnel passed the corner pier of the Hotel Belmont in solid rock, with only 10 ft. of rock between the concrete footing of the pier and the limit of excavation. The load on the footing was 850 tons. The two north-bound tracks of the Manhattan-Bronx Railway will be utilized for a shuttle train service between Broadway and the Queensborough Elevated Shaft. The extension of these tracks between Vanderbilt Avenue and the Elevator Shaft, and the ramp connecting the Elevator Shaft and the Grand Central Terminal are parts of the contract.

Route 43.

The principal features of interest on Route 43 are as follows:

(1) The design and construction of the "half-arch" section over the express tracks and the south-bound local near 39th and 38th Streets, respectively; (2) lowering the invert of the north-bound local track and maintaining traffic; (3) constructing the south-bound local and the express tracks beneath the old tunnels, and erecting steel at the latter places.

The Half-Arch Construction.—This construction was suggested by the contractor's engineers after the contract had been let. The original design is shown on Plate XL and Figs. 12 and 13. The specification stated that the contractor would be "required to allow and arrange with the Interborough Company to put in such cross-overs as may be needed between the existing tracks for the prosecution of the work. * * * To reconstruct or install and maintain such signals, lighting, and telephone cables as may be necessary for the proper operation of trains, as determined by the Engineer and the Interborough Company."

Cross-overs could have been made south of the new construction near 38th Street, but, in order to make any south of the station at 42d Street, an elaborate reconstruction of the existing structure would have been required.

On account of the cross-overs and their maintenance, and the great danger to the traveling public in the Subway due to temporary change of signals and track system, it was decided to eliminate all track changes.

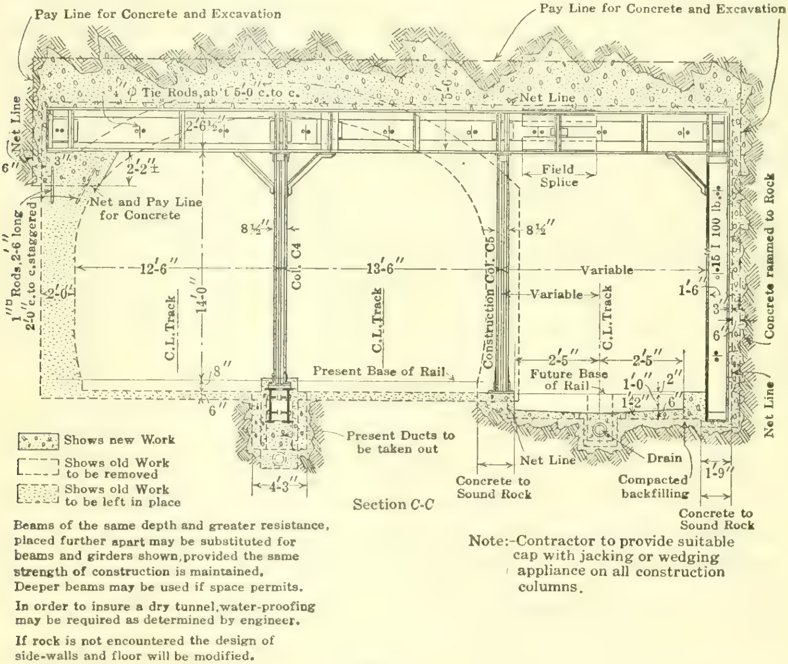
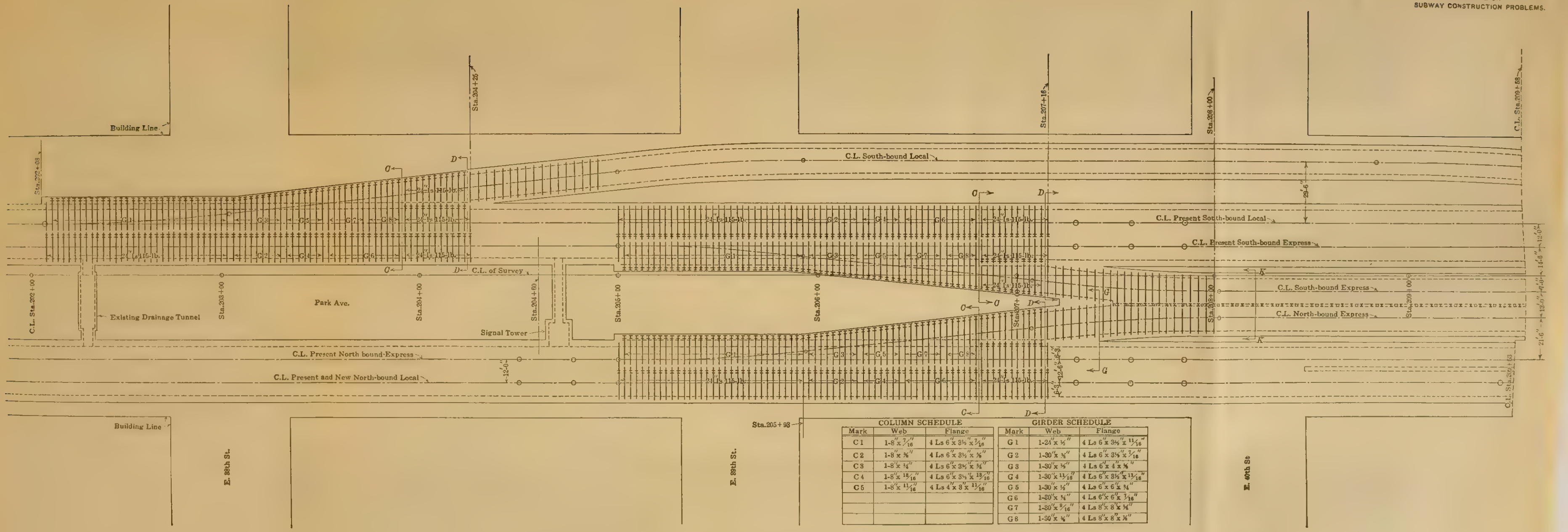


FIG. 12.

Drawings made in the contractor's field office, Figs. 14, 15, 16, and 17, show the several steps in the procedure for the proposed work. These plans contemplated removing one-half of the existing arch in short sections and erecting the new structure completely before removing another portion of the arch. The function of the new structure was to resist the crown thrust of the old arch and to support the rock above the new track. This plan omitted entirely any new work above the opposite tracks. In order to protect the trains and the tracks from any small pieces of falling material, a shield was designed for the



COLUMN SCHEDULE

Mark	Web	Flange
C 1	1-8″x 7/16″	4 Ls 6″x 3½″x 7/16″
C 2	1-8″x ⅜″	4 Ls 6″x 3½″x ⅜″
C 3	1-8″x ¾″	4 Ls 6″x 3½″x ¾″
C 4	1-8″x 13/16″	4 Ls 6″x 3½″x 13/16″
C 5	1-8″x 11/16″	4 Ls 4″x 3″x 11/16″

GIRDER SCHEDULE

Mark	Web	Flange
G 1	1-24″x ½″	4 Ls 6″x 3½″x 11/16″
G 2	1-30″x ⅜″	4 Ls 6″x 3½″x 7/16″
G 3	1-30″x ½″	4 Ls 6″x 4″x ⅝″
G 4	1-30″x 11/16″	4 Ls 6″x 3½″x 11/16″
G 5	1-30″x ½″	4 Ls 6″x 6″x ¾″
G 6	1-30″x ¾″	4 Ls 6″x 6″x 7/16″
G 7	1-30″x 9/16″	4 Ls 8″x 8″x ¾″
G 8	1-30″x ⅝″	4 Ls 8″x 8″x ⅜″

interior of the old tunnel and was erected before the work, above described, was commenced.

It was not claimed by the contractor, however, that the shield was designed to support large masses of falling concrete; it was built only to hold the concrete in place by fitting the ribs and lagging

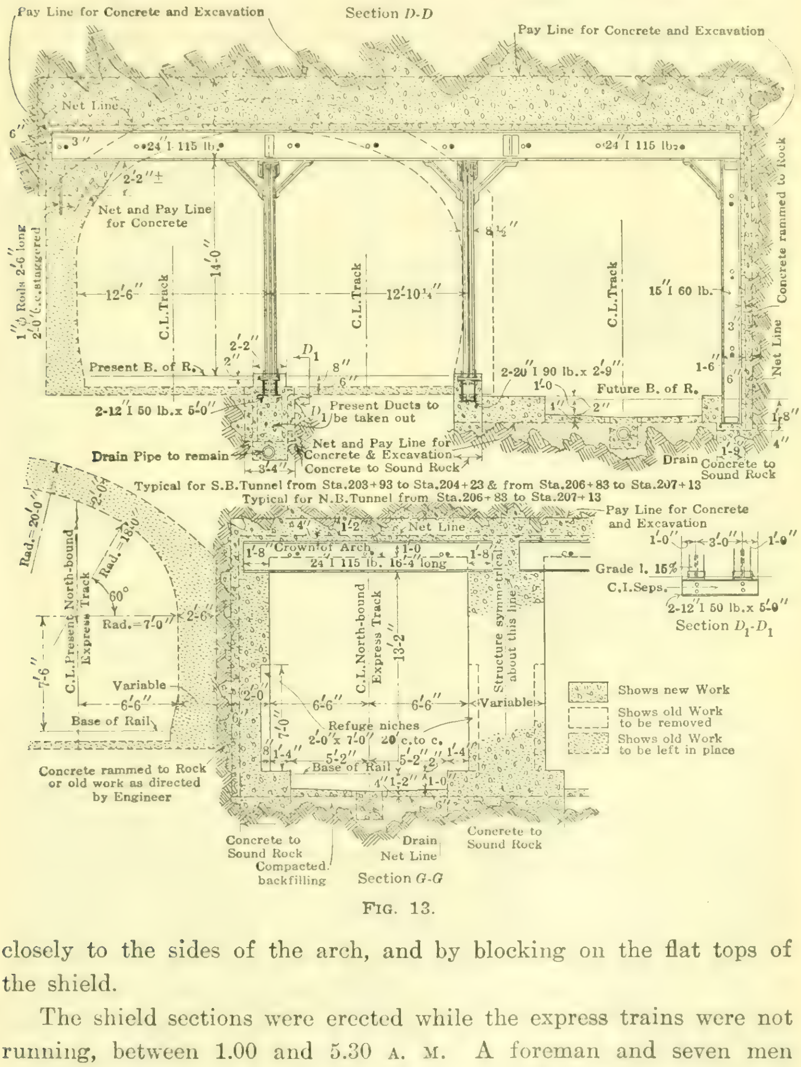
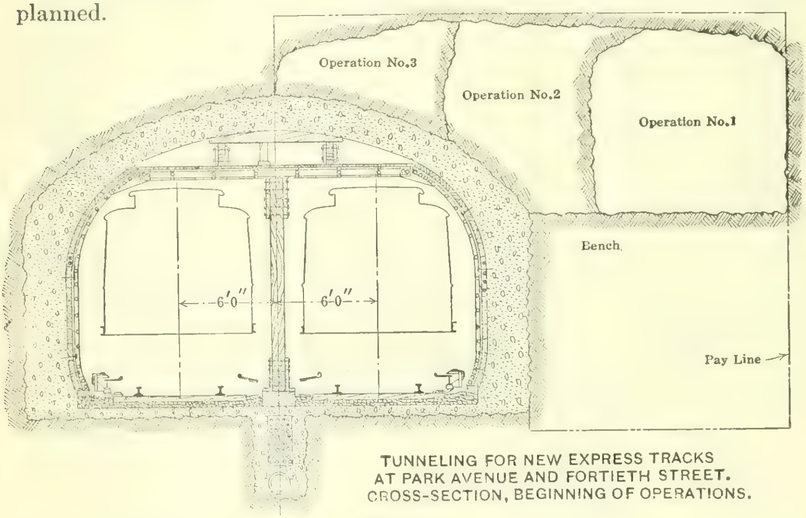


FIG. 13.

closely to the sides of the arch, and by blocking on the flat tops of the shield.

The shield sections were erected while the express trains were not running, between 1.00 and 5.30 A. M. A foreman and seven men constituted the gang that erected the shield. Rubber mats were laid

on the third-rail guards when the steel ribs were being erected. In order to insure the best possible organization, the men were instructed in their duties, such as placing horses for scaffolds, erection of the ribs, etc., as everything had to be removed and replaced for every train to pass. Consequently, it became necessary to put up a complete steel rib and fasten the parts together between trains running on the local tracks under a 7½-min. headway. The holes in the concrete arch for receiving the expansion bolts, which held the ribs, were drilled by another gang some time before the shields were placed. The different steps in the process of excavation were carried out as originally planned.



TUNNELING FOR NEW EXPRESS TRACKS
AT PARK AVENUE AND FORTIETH STREET.
CROSS-SECTION, BEGINNING OF OPERATIONS.

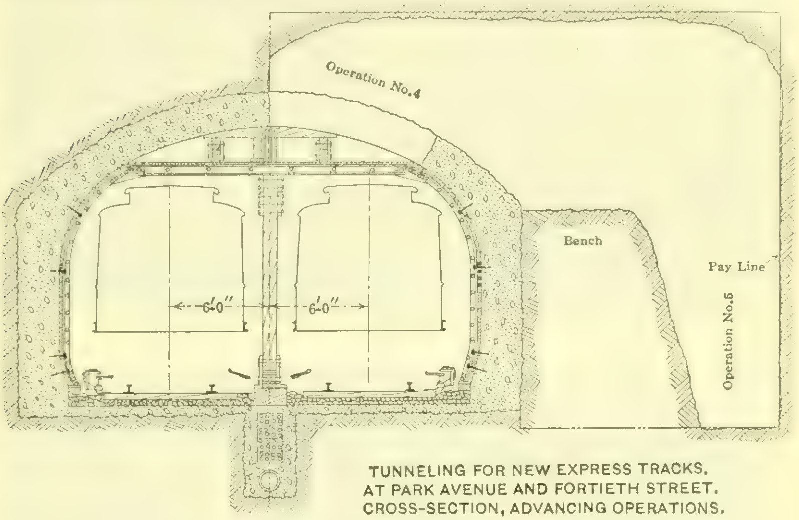
FIG. 14.

The heading shown as operation No. 1 on Fig. 14 was driven south from the 40th Street Shaft to the end of the reconstruction sections, but the heading for the half-arch work south of the south-bound local tunnel was driven about 50 ft. in advance of the widened section ahead of the steel erection. The drills used were Ingersoll-Rand jackhammers (BCR-430). In the heading 8-ft. holes were drilled and 6-ft. holes were driven for operations Nos. 2 and 3, perpendicular to the main heading. The bench was taken out in two lifts, as shown by Fig. 14. Where the concrete arch was thick enough to permit the use of explosives in light charges, this method was used. Otherwise, and for the greater portion of this work, the concrete was removed by plugs and feathers. The plugs were pieces of old drill steel formed into

wedges and driven by Ingersoll-Rand drills (BCR-53) from which the rotating device had been removed. The holes for plugging were drilled about 6 or 8 in. apart, and from 1 to 2 ft. from the face of the concrete to be removed.

The dynamite used was "Red Cross", low-freezing, 40% gelatine, 1¼ in. in diameter, one stick weighing about ½ lb.

The holes for operations Nos. 2 and 3 were about 12 in. from center to center, two being fired at one time with about one-quarter of a stick of dynamite in each hole.

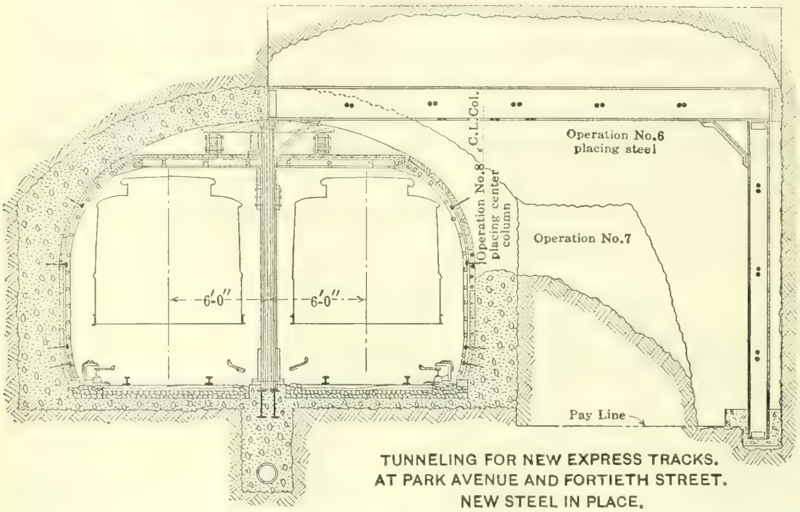


TUNNELING FOR NEW EXPRESS TRACKS, AT PARK AVENUE AND FORTIETH STREET. CROSS-SECTION, ADVANCING OPERATIONS.

FIG. 15.

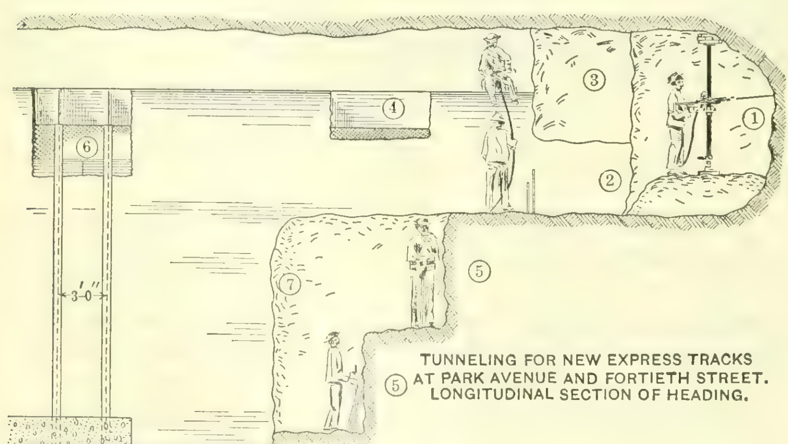
In the tunnels, near where the work was in progress the Interborough Company had stationed watchmen who were able to communicate with the tunnel foremen by telephone so that no blasts would be fired while trains were passing. After each shot the watchman examined the shield and track to see if everything was safe for the passage of trains. The roof-beams of the new construction are 3 ft. apart. Two beams were placed and concreted each week, and the necessary rock was excavated. The beams, which averaged 2 tons each, were lifted into place with a hoisting engine in the finished tunnel, rigged with a snatch block on the floor ahead and a fall fixed above the roof-beams in the rock by using a head-frame. The concrete forms for the roof were fastened to the roof-beams with hook-bolts. A timber bulkhead was built on the end beam and braced to the

rock face ahead. The concrete was mixed in a ½-cu. yd. mixer in the 40th Street Shaft above the tunnel roof, the materials being fed into the mixer from bins in the shaft above. The mixer discharged



TUNNELING FOR NEW EXPRESS TRACKS.
AT PARK AVENUE AND FORTIETH STREET.
NEW STEEL IN PLACE.

FIG. 16.



TUNNELING FOR NEW EXPRESS TRACKS
AT PARK AVENUE AND FORTIETH STREET.
LONGITUDINAL SECTION OF HEADING.

FIG. 17.

into cars running on a track above the tunnel floor high enough to permit men and muck cars to pass beneath, to and from the heading. Where the concrete was to be placed between and above the roof-beams a pair of timber guides was set up and a light special bucket

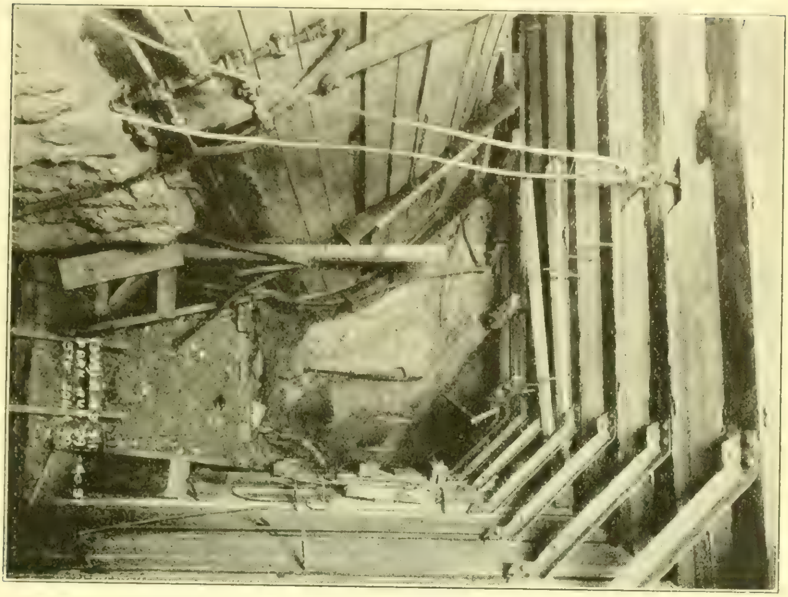


FIG. 18.—SHIELD WITH OLD ARCH REMOVED.

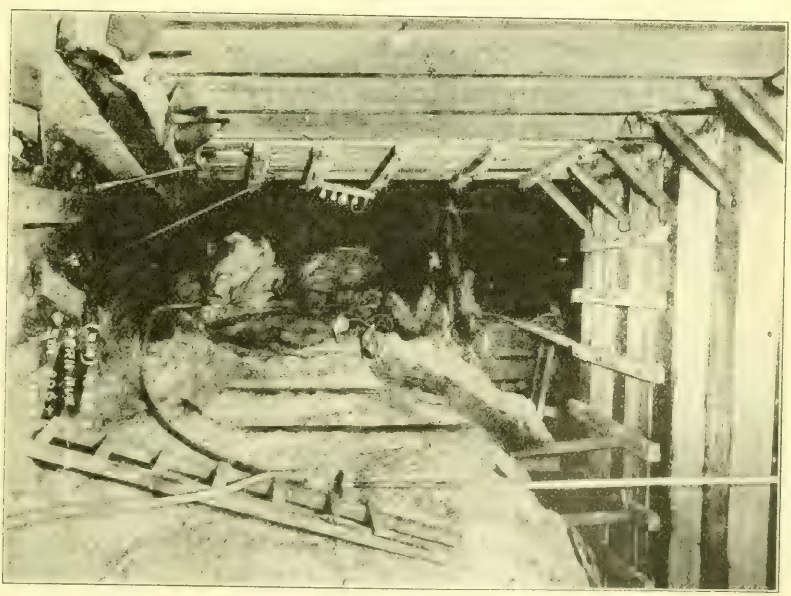


FIG. 19.—REMOVING BENCH.

for raising the concrete was run up the guides and dumped behind the beams.

LOWERING THE INVERT OF THE NORTH-BOUND LOCAL TRACK.

This work was between Stations 208 + 47 and 209 + 58, in the north-bound tunnel, where the old arch was not changed, and up to Station 210 + 18, which is near the south-east corner of 41st Street

CROSS-SECTION OF PARK AVENUE, SOUTH OF EAST 42D STREET, LOOKING NORTH.

The first bent of the new steel construction is shown in place over the old and new express tracks, and excavation is in progress over the north-bound tracks.

FIG. 20.

and Park Avenue. This latter portion is under new steel construction and is protected by shields like those used for the half-arch construction. The first step in this work was to remove a section of the third rail, after which the ballast was removed from the ends of the ties and 10-in. Bethlehem **H**-beams, with cover-plates, were fitted to the ends of the ties, the latter being supported on the bottom flanges of the beams. The beams were held in place with tie-rods. The

H-beams were 20 ft. long, and rested on the floor of the old tunnel. After the track had been supported in this manner, a pit was excavated between the tracks, deep enough to permit of working space for tunneling below the track. The cross-cut was made 10 ft. wide for the first one. The succeeding cuts were then excavated from the previous opening. As soon as one slice was completed the walls and invert were built and the H-beams were advanced for another increment. This work also included the underpinning of the side-wall of the tunnel arch south of Station 209 + 58, as shown on Plate XLII. The new wall footing was tied to the rock by steel rods set into drill holes and grouted.

This track, being for local service, was always operated both day and night. The express trains, however, did not run between 1.00 and 5.30 A. M. The temporary supports of the track were designed so that the new track could be laid beneath, for the greater portion of the section. Where the old and new tracks would interfere with each other, the new one will be omitted and the present one will be lowered into place and connected to the new one when the time comes to operate the Lexington Avenue Line.

The pay line north of Station 209 + 58 is similar to that shown on Plate XLII, with the exception that the contract plan contemplated excavating from the surface between the side-walls of the tunnel and the building line. The excavation, however, did not extend more than 2 ft. outside of the arch wall where the footing was renewed. Before the excavation for the north-bound local track was started, in the Grand Union Hotel lot, two piers of the building at the corner of 41st Street were underpinned to the sub-grade of the cut. Fig. 23 shows the concrete pier at the corner of the building.

The headings for the north-bound and south-bound express track construction north of 40th Street required no timbering. The old tunnels were heavily timbered south of 40th Street in places, and this timbering was all removed where it came within the reconstruction lines.

It was found that the grout of the old tunnel had completely filled the spaces between the concrete and the rock. The timbers of the old construction were completely surrounded by the concrete. Extraordinary precautions must have been taken in timbering the tunnel, as in many places where the reconstruction exposed the old

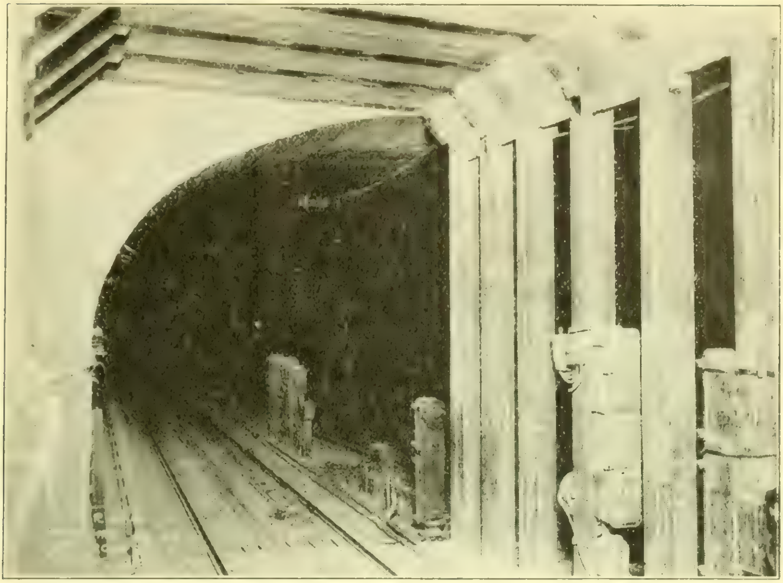


FIG. 21.—FINISHED PORTION OF HALF-ARCH CONSTRUCTION.

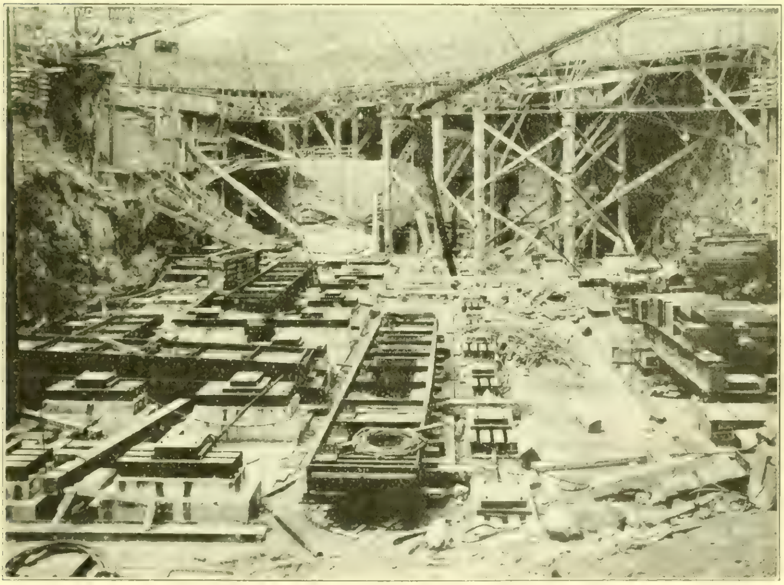


FIG. 22.—FLOOR OF DIAGONAL STATION, LARGE GIRDERS FOR EXPRESS TUNNEL LYING ON THE FLOOR.

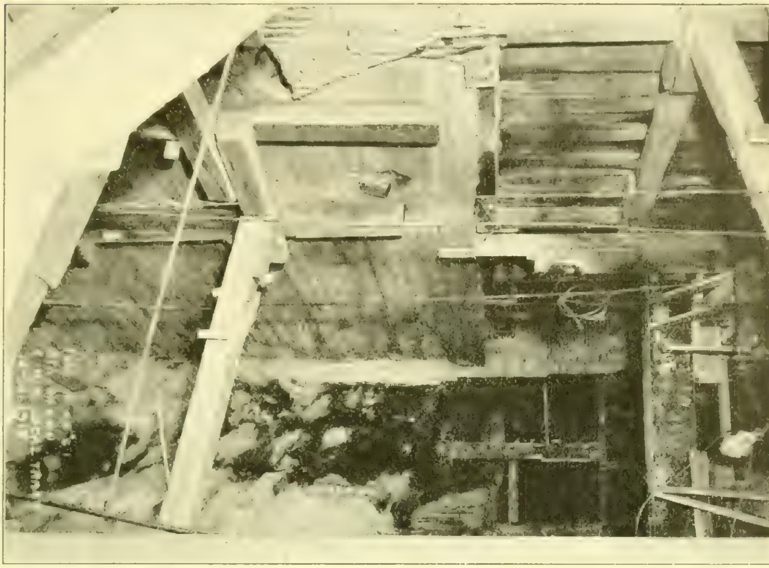
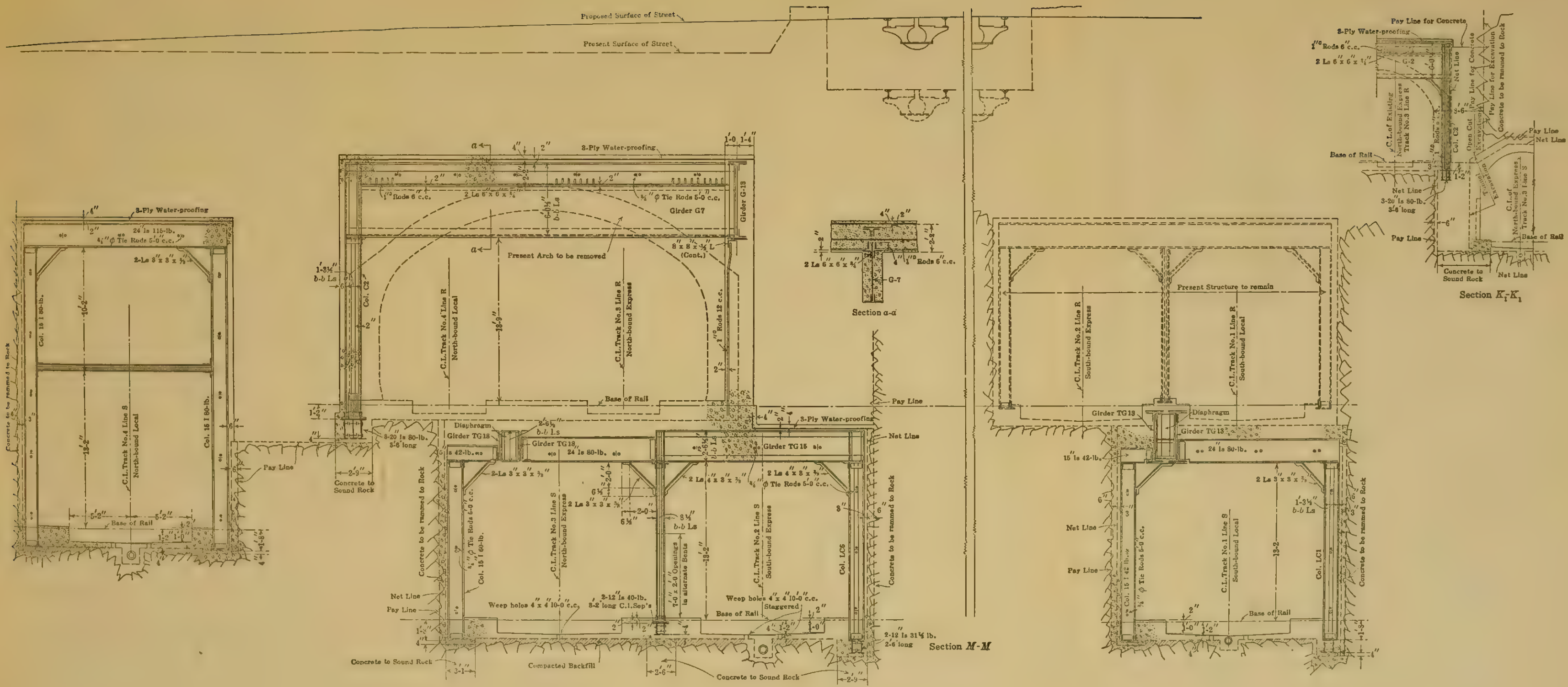


FIG. 23.—UNDERPINNING OF CORNER PIER AT 41ST STREET, AND CUT FOR NORTH-BOUND LOCAL.

FIG. 24.—SECTION OF DRIFT FOR EXPRESS TRACKS, LOOKING NORTH FROM 40TH STREET SHAFT IN "TWIN TUNNEL".

PLATE XLI.
PAPERS, AM. SOC. C. E.
NOVEMBER, 1917.
PERRINE ON
SUBWAY CONSTRUCTION PROBLEMS.



timber the rock was quite sound. Several places, however, were encountered where special care had to be exercised on account of disintegrated rock above the timbers.

Construction of the South-Bound Local and Express Tracks Under the Present Subway.

Plates XLI and XLII and Figs. 25 and 26 show the steel construction as planned, and on which the bids were based.

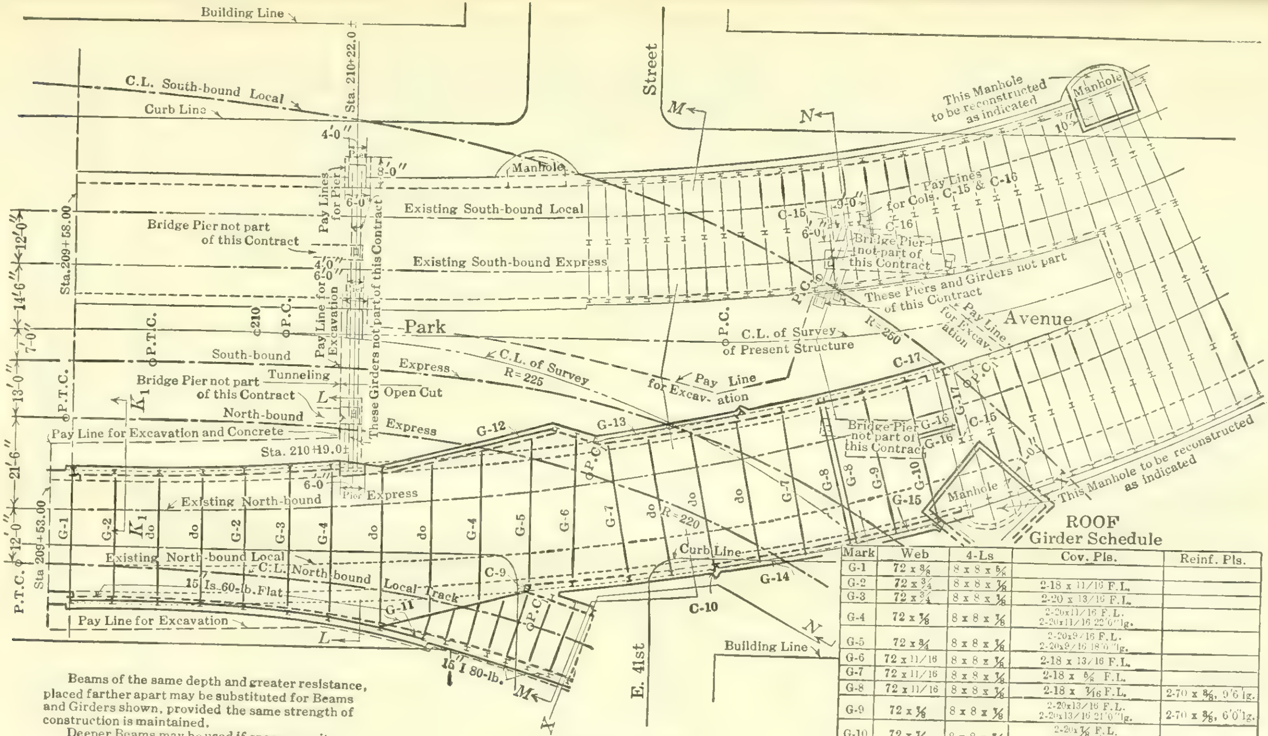
It will be noticed on Fig. 25 that the roof beams for the reconstruction of the north-bound tunnel are heavy plate girders spanning the two tracks. Some of these girders rest on columns made of I-beams built into the side-walls, and others are framed into longitudinal plate girders which are supported at the roof elevation by columns. (See the cross-section on Plate XLI.) The roof girders were only 12 or 15 in. above the tops of the cars. On account of the number and the great weight of these plate girders and the small space between the street deck supports, it was considered dangerous to attempt to erect the steel, as shown on the plans, above the Subway trains that pass this point day and night.

The engineers of the Commission were requested by the contractor to change the plate-girder design to the standard structure adopted on the original Subway, a piece of which is shown on Fig. 25, and, in section, on Plate XLI.

Girders G-12, G-13, G-14, and G-15 were to be placed over the roof of the new express tracks in the suggested plan, and were designed to support the side-wall columns. Two girders were designed to replace the four shown on Fig. 26 and marked TG-18 and TG-19.

An additional advantage was gained by placing the large girders at the roof level of the new express tunnel, for the reason that they were used to support the steel for the reconstruction of the present north-bound tunnel, thereby permitting the completion of the upper tunnel while the express tunnel was being built.

It was proposed to omit Girders TG-13 over the new south-bound local track and place the roof girders radially with the center line of the track. Drifts were driven under the present Subway for the express and south-bound local. The drifts were 9 ft. high and from 9 to 12 ft. wide, and were timbered where they were directly below the track floors of the Subway. There were from 6 to 9 ft. of rock



Beams of the same depth and greater resistance, placed farther apart may be substituted for Beams and Girders shown, provided the same strength of construction is maintained.

Deeper Beams may be used if space permits.

In order to insure a Dry Tunnel, Water-proofing may be required, as determined by Engineer.

If Rock is not encountered the design of Side-walls and Floor will be Modified.

ROOF PLAN
LEGEND

Present Track to remain shown thus:
" " abondoned " "
New " " "
Present material to remain, " " { seen / hidden
" " be removed " "
New " " { seen / hidden

ROOF Girder Schedule

Mark	Web	4-Ls	Cov. Pls.	Reinf. Pls.
G-1	72 x 3/8	8 x 8 x 5/8		
G-2	72 x 3/4	8 x 8 x 7/8	2-18 x 11/16 F.L.	
G-3	72 x 3/4	8 x 8 x 7/8	2-20 x 13/16 F.L.	
G-4	72 x 7/8	8 x 8 x 7/8	2-20x11/16 F.L. 2-20x11/16 22'0" lg.	
G-5	72 x 3/4	8 x 8 x 7/8	2-20x9/16 F.L. 2-20x9/16 18'0" lg.	
G-6	72 x 11/16	8 x 8 x 7/8	2-18 x 13/16 F.L.	
G-7	72 x 11/16	8 x 8 x 7/8	2-18 x 5/8 F.L.	
G-8	72 x 11/16	8 x 8 x 7/8	2-18 x 7/16 F.L.	2-70 x 3/8, 9'6" lg.
G-9	72 x 7/8	8 x 8 x 7/8	2-20x13/16 F.L. 2-20x13/16 21'0" lg.	2-70 x 3/8, 6'0" lg.
G-10	72 x 7/8	8 x 8 x 7/8	2-20x7/8 F.L. 2-20x7/8 22'0" lg.	
G-11	2-72 x 5/8	8 x 8 x 7/8	2-20x3/4 F.L., 2-20x3/4 18' lg. 2-20x3/4 25' lg., 2-20x3/4 30' lg.	
G-12	2-72 x 11/16	8 x 8 x 7/8	2-20x7/8 F.L., 2-20x7/8 20' lg. 2-20x7/8 28' lg., 2-20x7/8 33' lg.	
G-13	72 x 11/16	8 x 8 x 7/8	2-20x7/8 F.L. 2-20x7/8 2[illegible]'0" lg.	4-70 x 3/8, 5'0" lg.
G-14	72 x 3/4	8 x 8 x 7/8	2-20 x 7/8 F.L.	
G-15	72 x 11/16	8 x 8 x 7/8	2-20 x 3/4 F.L.	2-70 x 3/8, 7'0" lg.
G-16	36 x 13/16	6 x 6 x 5/8		
G-17	72 x 13/16	6 x 6 x 5/8		

FIG. 25.

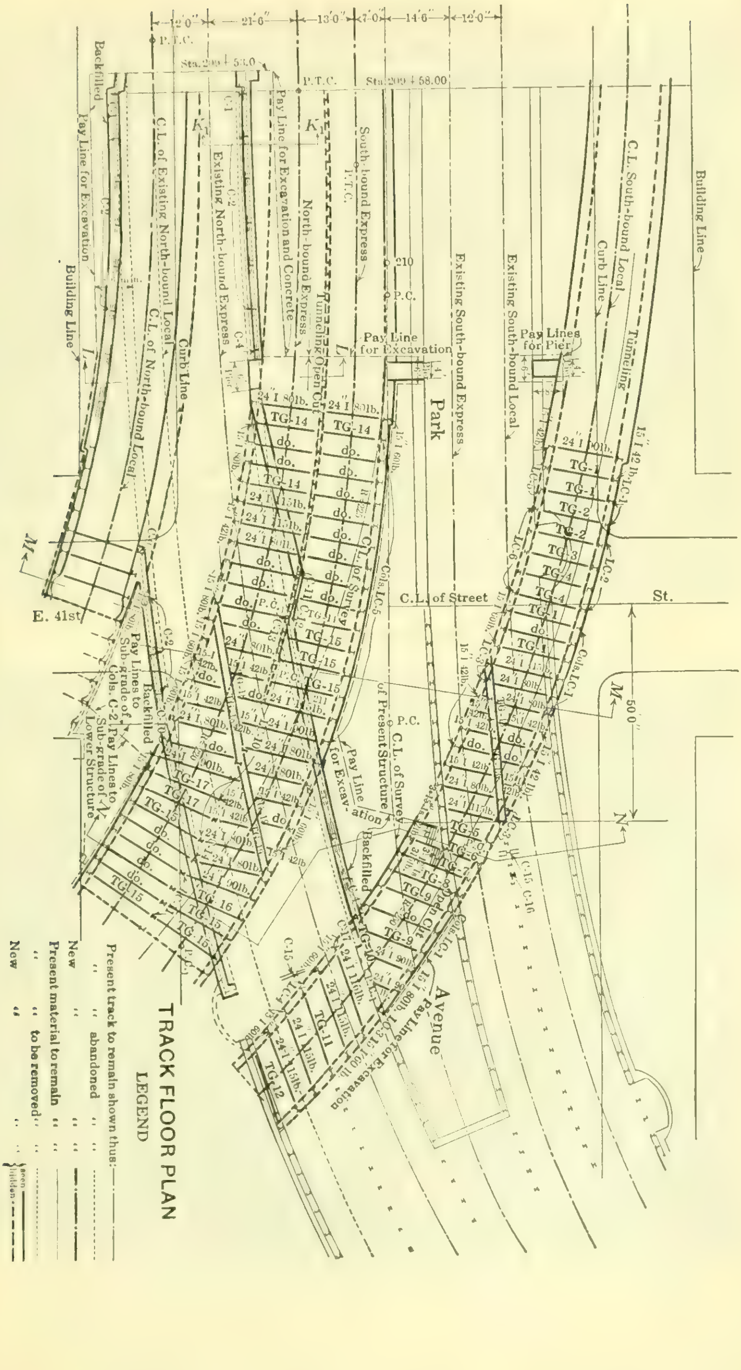

TRACK FLOOR PLAN
LEGEND
Present track to remain shown thus:
" " abandoned " "
New " "
Present material to remain " "
" " to be removed " "
New " "
seen
hidden
C.L. of Existing North-bound Local
C.L. of North-bound Local
Existing North-bound Express
North-bound Express
South-bound Express
Existing South-bound Express
Existing South-bound Local
C.L. South-bound Local
Pay Line for Excavation
Pay Line for Excavation and Concrete
Pay Lines for Pier
Tunneling
Open Cut
Building Line
Curb Line
Backfilled
C.L. of Street
C.L. of Survey
C.L. of Present Structure
Park
Avenue
E. 41st
St.
Sta. 209 + 58.00
P.T.C.
P.C.
50′ 0″

Fig. 26.

left between the roof of the drift and the bottom of the track floor. The drifts were enlarged by men working from scaffolds, using jack-hammer drills. A band of rock was excavated to receive one or two bents of steel, and these were at once erected and concreted. This operation was carried on in four places by two gangs, each alternating between two points in order to allow the concrete to set while drilling for a new band. The contract plans contemplated the removal of the concrete floor of the old tunnels, in order to place the new steel, but, by the adopted plan, it was possible to place the new steel without disturbing the invert (excepting where deep girders occurred) or interfering with the Subway traffic. As an extra precaution, the tracks of the Subway were supported by Bethlehem **H**-beams at the ends of the ties, and these were placed so as to span the space where the rock was being removed below. The columns of the Subway structure were also supported temporarily by the underpinning spanning the cut.

When the drift of the south-bound local was past the Subway structure, it was widened to the full size of the new sections, and advanced, south, by driving a top heading and removing the bench at the same time. The express drift, after passing the Subway structure, met the heading that was driven north from the 40th Street Shaft. After the old structure had been supported on the roof of the new one, grout was pumped into the joint between them.

Erecting Large Girders and Columns for the Express Tunnels.

The first operation in connection with the erection of the steel was to excavate the trenches for the girders, and sink pits in the rock, from the level of the Subway to the sub-grade of the grillages. Seven of these columns were just outside of the walls of the old north-bound tunnel and three were between the Subway tracks. The large girders that were at the east of the old Subway at the points where Girders G-14 and G-15 are shown on Fig. 25, were raised into position from the open cut with a gin-pole and tackle and rolled into place. All the other large girders were run through the express track drift from the station floor, and south in the express tunnel far enough to permit them to be pulled endwise up into the open excavation at the west side of the north-bound tunnel. The two girders between the tracks were passed, from the express cut, through an

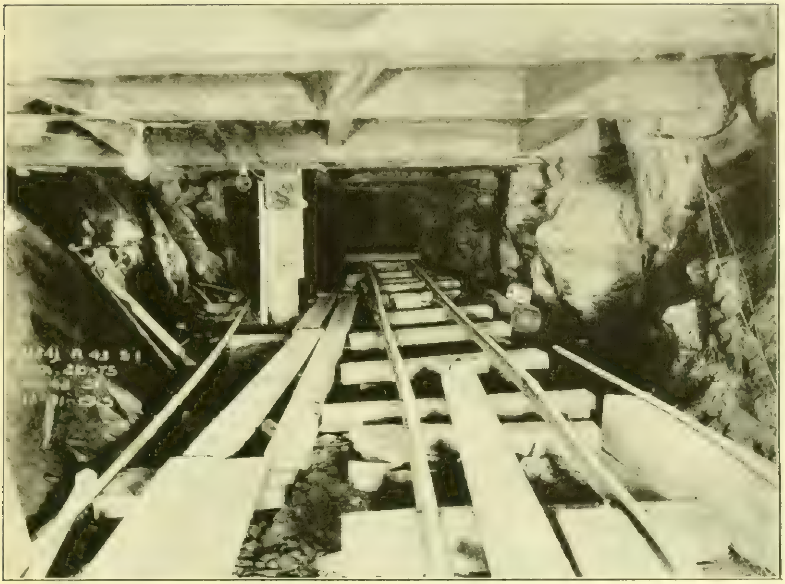


FIG. 27.—EXPRESS DRIFT, LOOKING SOUTH.

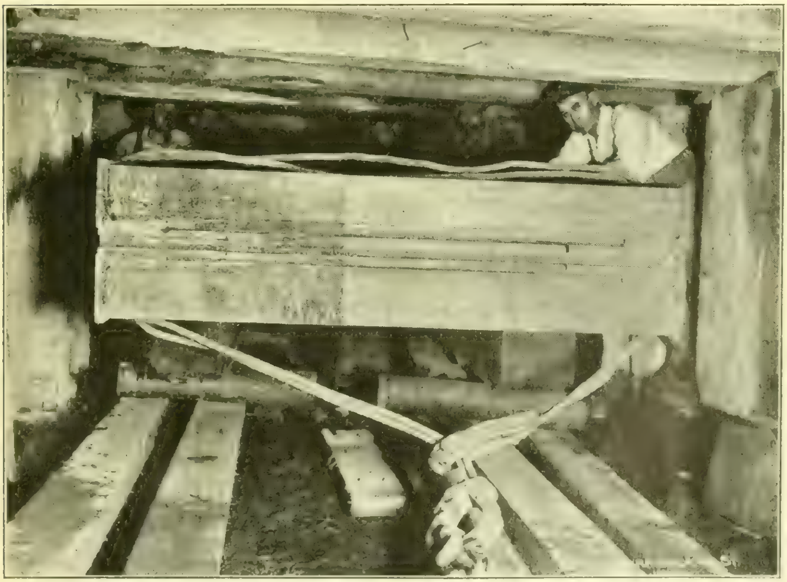


FIG. 28.—EXPRESS DRIFT WITH LARGE GIRDER PASSING THROUGH.

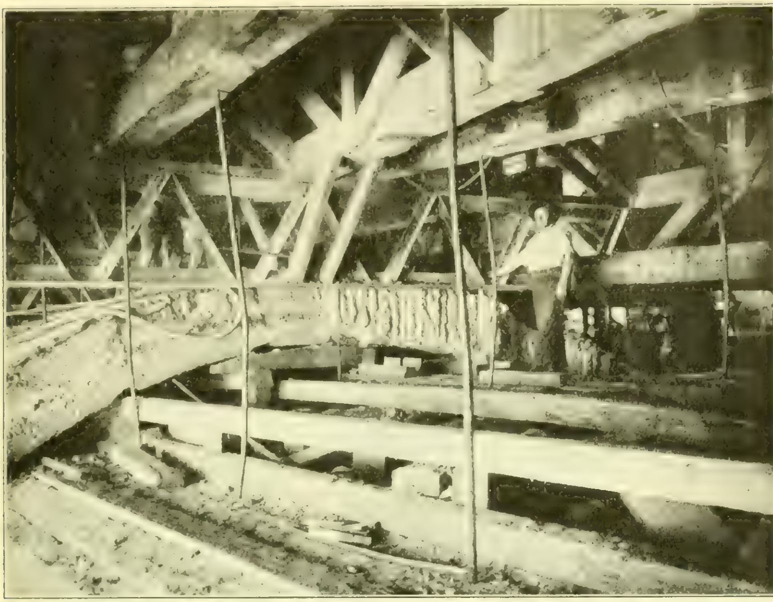


FIG. 29.—ARCH OF NORTH-BOUND TUNNEL DRILLED AND REMOVED WITH PLUGS AND FEATHERS.

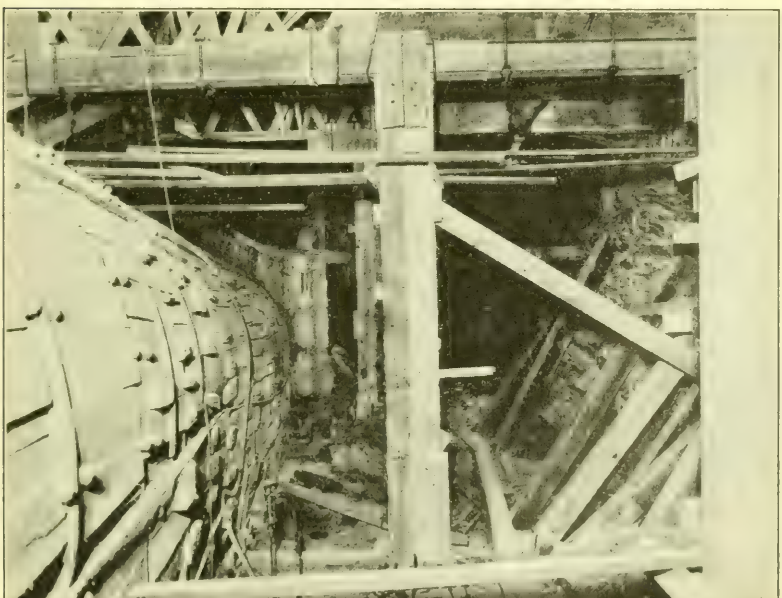
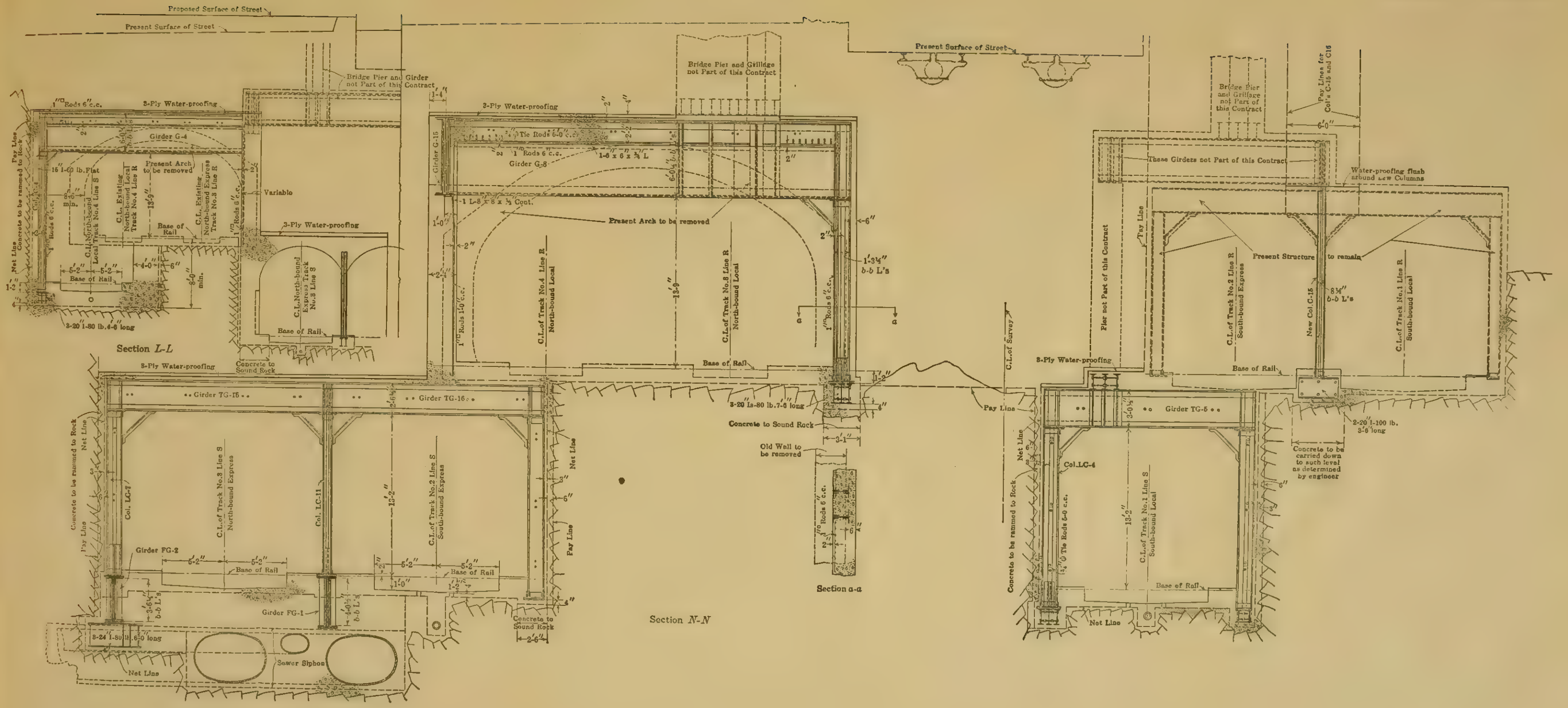
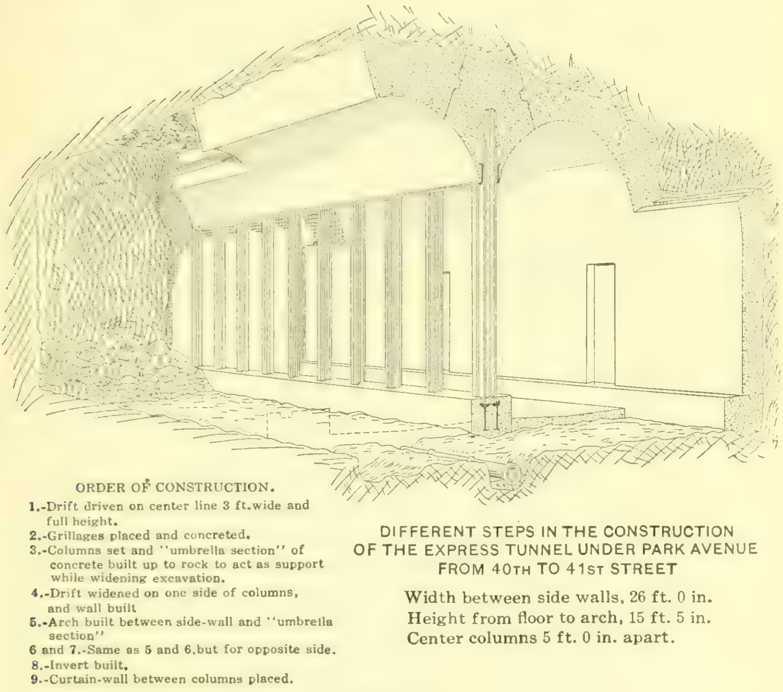


FIG. 30.—EXPRESS CUT. DRILLING FOR TRENCH TO RECEIVE LARGE GIRDERS.


Proposed Surface of Street
Present Surface of Street
Bridge Pier and Girder not Part of this Contract
Bridge Pier and Grillage not Part of this Contract
Present Surface of Street
Bridge Pier and Grillage not Part of this Contract
Pay Lines for Col's C-15 and C16
3-Ply Water-proofing
Girder G-4
Present Arch to be removed
Variable
Base of Rail
Section L-L
Girder G-8
Present Arch to be removed
C.L.of Track No.4 Line R North-bound Local
C.L.of Track No.3 Line R North-bound Local
These Girders not Part of this Contract
Water-proofing flush around new Columns
Present Structure to remain
C.L.of Track No.2 Line R South-bound Express
C.L.of Track No.1 Line R South-bound Local
New Col.C-15
Pier not Part of this Contract
C.L.of Survey
Concrete to Sound Rock
Old Wall to be removed
Section a-a
Girder TG-15
Girder TG-16
Girder TG-5
C.L.of Track No.3 Line S North-bound Express
C.L.of Track No.2 Line S South-bound Express
C.L.of Track No.1 Line S South-bound Local
Col. LC-7
Col. LC-11
Col.LC-4
Girder FG-2
Girder FG-1
Sewer Siphon
Net Line
Pay Line
Concrete to be rammed to Rock
Concrete to be carried down to such level as determined by engineer
Section N-N

opening in the shield, across the express track, and placed at night when the express service was not in operation.

The trench between the tracks and the pits, for the columns that were to support the large girders, was ready some time before the girders arrived. Several of the pits were stoped up from the express drifts, and others were sunk from the bottom of the open cut and the tunnel floor.



ORDER OF CONSTRUCTION.

1.-Drift driven on center line 3 ft. wide and full height.
2.-Grillages placed and concreted.
3.-Columns set and "umbrella section" of concrete built up to rock to act as support while widening excavation.
4.-Drift widened on one side of columns, and wall built
5.-Arch built between side-wall and "umbrella section"
6 and 7.-Same as 5 and 6, but for opposite side.
8.-Invert built.
9.-Curtain-wall between columns placed.

DIFFERENT STEPS IN THE CONSTRUCTION OF THE EXPRESS TUNNEL UNDER PARK AVENUE FROM 40TH TO 41ST STREET

Width between side walls, 26 ft. 0 in.
Height from floor to arch, 15 ft. 5 in.
Center columns 5 ft. 0 in. apart.

FIG. 33.

The weights of the girders west of the tracks are 70 350 and 28 900 lb.; those between the tracks, 36 500 and 20 500 lb.; and the two east of the tracks, 16 600 and 15 200 lb.

Personnel.—The personnel was as follows: Representing the Public Service Commission of the First District of New York: Alfred Craven, M. Am. Soc. C. E., Chief Engineer; Robert Ridgway, M. Am. Soc. C. E., Engineer of Subway Construction; John H. Myers, Assoc. M. Am. Soc. C. E., Engineer, Second Division; Mr. Stephen Schmidt, Assistant Engineer; and Mr. Melvin Miller, Assistant Engineer. Rep-

resenting the Rapid Transit Subway Construction Company, Contractor: George H. Pegram, President, Am. Soc. C. E., Chief Engineer; Robert A. Shailer, M. Am. Soc. C. E., Tunnel Engineer; George Perrine, M. Am. Soc. C. E., Assistant Tunnel Engineer; and Mr. Thomas McCormick, General Superintendent.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

SPECIFICATIONS AND METHODS OF TESTS FOR PORTLAND CEMENT

The following specifications and methods of tests for Portland cement are those recommended by the Joint Conference on Uniform Methods of Tests and Standard Specifications for Cement. The Conference was created for the purpose of securing uniformity and reconciling differences in cement specifications and methods of tests. The membership of the Conference was constituted as follows:

AMERICAN SOCIETY OF CIVIL ENGINEERS

Alfred Noble*
George S. Webster
Richard L. Humphrey

AMERICAN SOCIETY FOR TESTING MATERIALS

George F. Swain
Olaf Hoff
Clifford Richardson

UNITED STATES GOVERNMENT

Arthur P. Davis
Asa E. Phillips
Rudolph J. Wig

The Conference was organized on October 24th, 1912. Many data relating to the subjects under consideration were gathered from manufacturers, users, laboratories, and individuals. In addition, many original data were secured from experiments and tests conducted by the Conference or under its direction. The results were correlated and studied, and these, together with the conclusions based upon them, were published by the Conference in reports dated April 28th, 1915, and June 1st, 1916.

In final form, as here presented, the specifications and methods of tests were transmitted to the Society on January 16th, 1917.

* Deceased.

SPECIFICATIONS.

Definition. *1.*—Portland cement is the product obtained by finely pulverizing clinker produced by calcining to incipient fusion, an intimate and properly proportioned mixture of argillaceous and calcareous materials, with no additions subsequent to calcination excepting water and calcined or uncalcined gypsum.

I.—Chemical Properties.

Chemical Limits. *2*—The following limits shall not be exceeded:

Loss on ignition, per cent	4.00
Insoluble residue, per cent	0.85
Sulfuric anhydride (SO_3), per cent	2.00
Magnesia (MgO), per cent	5.00

II.—Physical Properties.

Specific Gravity. *3.*—The specific gravity of cement shall be not less than 3.10 (3.07 for white Portland cement). Should the test of cement as received fall below this requirement a second test may be made upon an ignited sample. The specific gravity test will not be made unless specifically ordered.

Fineness. *4.*—The residue on a standard No. 200 sieve shall not exceed 22 per cent. by weight.

Soundness. *5.*—A pat of neat cement shall remain firm and hard, and show no signs of distortion, cracking, checking, or disintegration in the steam test for soundness.

Time of Setting. *6.*—The cement shall not develop initial set in less than 45 minutes when the Vicat needle is used or 60 minutes when the Gillmore needle is used. Final set shall be attained within 10 hours.

Tensile Strength. *7.*—The average tensile strength in pounds per square inch of not less than three standard mortar briquettes (see Section 51) composed of one part cement and three parts standard sand, by weight, shall be equal to or higher than the following:

Age at test, in days.	Storage of briquettes.	Tensile stength, in pounds per square inch.
7	1 day in moist air, 6 days in water	200
28	1 day in moist air, 27 days in water	300

8.—The average tensile strength of standard mortar at 28 days shall be higher than the strength at 7 days.

III.—PACKAGES, MARKING, AND STORAGE.

9.—The cement shall be delivered in suitable bags or barrels with the brand and name of the manufacturer plainly marked thereon, unless shipped in bulk. A bag shall contain 94 lb. net. A barrel shall contain 376 lb. net. **Packages and Marking.**

10.—The cement shall be stored in such a manner as to permit easy access for proper inspection and identification of each shipment, and in a suitable weather-tight building which will protect the cement from dampness. **Storage.**

IV.—INSPECTION.

11.—Every facility shall be provided the purchaser for careful sampling and inspection at either the mill or at the site of the work, as may be specified by the purchaser. At least 10 days from the time of sampling shall be allowed for the completion of the 7-day test, and at least 31 days shall be allowed for the completion of the 28-day test. The cement shall be tested in accordance with the methods hereinafter prescribed. The 28-day test shall be waived only when specifically so ordered. **Inspection.**

V.—REJECTION.

12.—The cement may be rejected if it fails to meet any of the requirements of these specifications. **Rejection.**

13.—Cement shall not be rejected on account of failure to meet the fineness requirement if upon retest after drying at 100° C. for one hour it meets this requirement.

14.—Cement failing to meet the test for soundness in steam may be accepted if it passes a retest using a new sample at any time within 28 days thereafter.

15.—Packages varying more than 5 per cent. from the specified weight may be rejected; and if the average weight of packages in any shipment, as shown by weighing 50 packages taken at random, is less than that specified, the entire shipment may be rejected.

TESTS.

VI.—SAMPLING.

16.—Tests may be made on individual or composite samples as may be ordered. Each test sample should weigh at least 8 lb. **Number of Samples.**

17.—(*a*) *Individual Sample.*—If sampled in cars one test sample shall be taken from each 50 bbl. or fraction thereof. If sampled in bins one sample shall be taken from each 100 bbl.

(*b*) *Composite Sample.*—If sampled in cars one sample shall be taken from one sack in each 40 sacks (or 1 bbl. in each 10 bbl.) and combined to form one test sample. If sampled in bins or warehouses one test sample shall represent not more than 200 bbl.

Method of Sampling.

18.—Cement may be sampled at the mill by any of the following methods that may be practicable, as ordered:

(*a*) *From the Conveyor Delivering to the Bin.*—At least 8 lb. of cement shall be taken from approximately each 100 bbl. passing over the conveyor.

(*b*) *From Filled Bins by Means of Proper Sampling Tubes.*—Tubes inserted vertically may be used for sampling cement to a maximum depth of 10 ft. Tubes inserted horizontally may be used where the construction of the bin permits. Samples shall be taken from points well distributed over the face of the bin.

(*c*) *From Filled Bins at Points of Discharge.*—Sufficient cement shall be drawn from the discharge openings to obtain samples representative of the cement contained in the bin, as determined by the appearance at the discharge openings of indicators placed on the surface of the cement directly above these openings before drawing of the cement is started.

Treatment of Sample.

19.—Samples preferably shall be shipped and stored in air-tight containers. Samples shall be passed through a sieve having 20 meshes per linear inch in order to thoroughly mix the sample, break up lumps and remove foreign materials.

VII.—Chemical Analysis.

Loss on Ignition.

Method.

20.—One gram of cement shall be heated in a weighed covered platinum crucible, of 20 to 25-cc. capacity, as follows, using either method (*a*) or (*b*) as ordered:

(*a*) The crucible shall be placed in a hole in an asbestos board, clamped horizontally so that about three-fifths of the crucible projects below, and blasted at a full red heat for 15 minutes with an inclined flame; the loss in weight shall be checked by a second blasting for 5 minutes. Care shall be taken to wipe off particles of asbestos that may adhere to the crucible when withdrawn from the hole in the board. Greater neatness and shortening of the time of heating are secured by making a hole to fit the crucible in a circular disk of sheet platinum and placing this disk over a somewhat larger hole in an asbestos board

(*b*) The crucible shall be placed in a muffle at any temperature between 900 and 1 000° C. for 15 minutes and the loss in weight shall be checked by a second heating for 5 minutes.

Permissible Variation.

21.—A permissible variation of 0.25 will be allowed, and all results in excess of the specified limit but within this permissible variation shall be reported as 4 per cent.

Insoluble Residue.

Method.

22.—To a 1-g. sample of cement shall be added 10 cc. of water and 5 cc. of concentrated hydrochloric acid; the liquid shall be warmed

until effervescence ceases. The solution shall be diluted to 50 cc. and digested on a steam bath or hot plate until it is evident that decomposition of the cement is complete. The residue shall be filtered, washed with cold water, and the filter paper and contents digested in about 30 cc. of a 5-per-cent. solution of sodium carbonate, the liquid being held at a temperature just short of boiling for 15 minutes. The remaining residue shall be filtered, washed with cold water, then with a few drops of hot hydrochloric acid, 1: 9, and finally with hot water, and then ignited at a red heat and weighed as the insoluble residue.

23.—A permissible variation of 0.15 will be allowed, and all results in excess of the specified limit but within this permissible variation shall be reported as 0.85 per cent. Permissible Variation.

Sulfuric Anhydride.

24.—One gram of the cement shall be dissolved in 5 cc. of concentrated hydrochloric acid diluted with 5 cc. of water, with gentle warming; when solution is complete 40 cc. of water shall be added, the solution filtered, and the residue washed thoroughly with water. The solution shall be diluted to 250 cc., heated to boiling and 10 cc. of a hot 10-per-cent. solution of barium chloride shall be added slowly, drop by drop, from a pipette and the boiling continued until the precipitate is well formed. The solution shall be digested on the steam bath until the precipitate has settled. The precipitate shall be filtered, washed, and the paper and contents placed in a weighed platinum crucible and the paper slowly charred and consumed without flaming. The barium sulfate shall then be ignited and weighed. The weight obtained multiplied by 34.3 gives the percentage of sulfuric anhydride. The acid filtrate obtained in the determination of the insoluble residue may be used for the estimation of sulfuric anhydride instead of using a separate sample. Method.

25.—A permissible variation of 0.10 will be allowed, and all results in excess of the specified limit but within this permissible variation shall be reported as 2.00 per cent. Permissible Variation.

Magnesia.

26.—To 0.5 g. of the cement in an evaporating dish shall be added 10 cc. of water to prevent lumping and then 10 cc. of concentrated hydrochloric acid. The liquid shall be gently heated and agitated until attack is complete. The solution shall then be evaporated to complete dryness on a steam or water bath. To hasten dehydration the residue may be heated to 150 or even 200° C. for one-half to one hour. The residue shall be treated with 10 cc. of concentrated hydrochloric acid diluted with an equal amount of water. The dish shall be covered and the solution digested for ten minutes on a steam bath or water Method.

bath. The diluted solution shall be filtered and the separated silica washed thoroughly with water.[1] Five cubic centimeters of concentrated hydrochloric acid and sufficient bromine water to precipitate any manganese which may be present, shall be added to the filtrate (about 250 cc.). This shall be made alkaline with ammonium hydroxide, boiled until there is but a faint odor of ammonia, and the precipitated iron and aluminum hydroxides, after settling, shall be washed with hot water, once by decantation and slightly on the filter. Setting aside the filtrate, the precipitate shall be transferred by a jet of hot water to the precipitating vessel and dissolved in 10 cc. of hot hydrochloric acid. The paper shall be extracted with acid, the solution and washings being added to the main solution. The aluminum and iron shall then be reprecipitated at boiling heat by ammonium hydroxide and bromine water in a volume of about 100 cc., and the second precipitate shall be collected and washed on the filter used in the first instance if this is still intact. To the combined filtrates from the hydroxides of iron and aluminum, reduced in volume if need be, 1 cc. of ammonium hydroxide shall be added, the solution brought to boiling, 25 cc. of a saturated solution of boiling ammonium oxalate added, and the boiling continued until the precipitated calcium oxalate has assumed a well-defined granular form. The precipitate after one hour shall be filtered and washed, then with the filter shall be placed wet in a platinum crucible, and the paper burned off over a small flame of a Bunsen burner; after ignition it shall be redissolved in hydrochloric acid and the solution diluted to 100 cc. Ammonia shall be added in slight excess, and the liquid boiled. The lime shall then be reprecipitated by ammonium oxalate, allowed to stand until settled, filtered and washed. The combined filtrates from the calcium precipitates shall be acidified with hydrochloric acid, concentrated on the steam bath to about 150 cc., and made slightly alkaline with ammonium hydroxide, boiled and filtered (to remove a little aluminum and iron and perhaps calcium). When cool, 10 cc. of saturated solution of sodium-ammonium-hydrogen phosphate shall be added with constant stirring. When the crystallin ammonium-magnesium orthophosphate has formed, ammonia shall be added in moderate excess. The solution shall be set aside for several hours in a cool place, filtered and washed with water containing 2.5 per cent. of NH_3. The precipitate shall be dissolved in a small quantity of hot hydrochloric acid, the solution diluted to about 100 cc., 1 cc. of a saturated solution of sodium-ammonium-hydrogen phosphate added, and ammonia drop by drop, with constant stirring, until the precipitate is again formed as described and the ammonia is in moderate excess. The precipitate shall then be allowed to stand about two hours, filtered and washed as before. The paper

[1] Since this procedure does not involve the determination of silica, a second evaporation is unnecessary.

and contents shall be placed in a weighed platinum crucible, the paper slowly charred, and the resulting carbon carefully burned off. The precipitate shall then be ignited to constant weight over a Meker burner, or a blast not strong enough to soften or melt the pyrophosphate. The weight of magnesium pyrophosphate obtained multiplied by 72.5 gives the percentage of magnesia. The precipitate so obtained always contains some calcium and usually small quantities of iron, aluminum, and manganese as phosphates.

27.—A permissible variation of 0.4 will be allowed, and all results in excess of the specified limit but within this permissible variation shall be reported at 5.00 per cent. **Permissible Variation.**

VIII.—Determination of Specific Gravity.

28.—The determination of specific gravity shall be made with a standardized Le Chatelier apparatus which conforms to the requirements illustrated in Fig. 1. This apparatus is standardized by the United States Bureau of Standards. Kerosene free from water, or benzine not lighter than 62° Baumé, shall be used in making this determination. **Apparatus.**

29.—The flask shall be filled with either of these liquids to a point on the stem between zero and one cubic centimeter, and 64 g. of cement, of the same temperature as the liquid, shall be slowly introduced, taking care that the cement does not adhere to the inside of the flask above the liquid and to free the cement from air by rolling the flask in an inclined position. After all the cement is introduced, the level of the liquid will rise to some division of the graduated neck; the difference between readings is the volume displaced by 64 g. of the cement. **Method.**

The specific gravity shall then be obtained from the formula

$$\text{Specific gravity} = \frac{\text{Weight of cement (g.)}}{\text{Displaced volume (cc.)}}$$

30.—The flask, during the operation, shall be kept immersed in water, in order to avoid variations in the temperature of the liquid in the flask, which shall not exceed 0°.5 C. The results of repeated tests should agree within 0.01.

31.—The determination of specific gravity shall be made on the cement as received; if it falls below 3.10, a second determination shall be made after igniting the sample as described in Section 20.

IX.—Determination of Fineness.

32.—Wire cloth for standard sieves for cement shall be woven (not twilled) from brass, bronze, or other suitable wire, and mounted without distortion on frames not less than 1½ in. below the top of the frame. **Apparatus.**

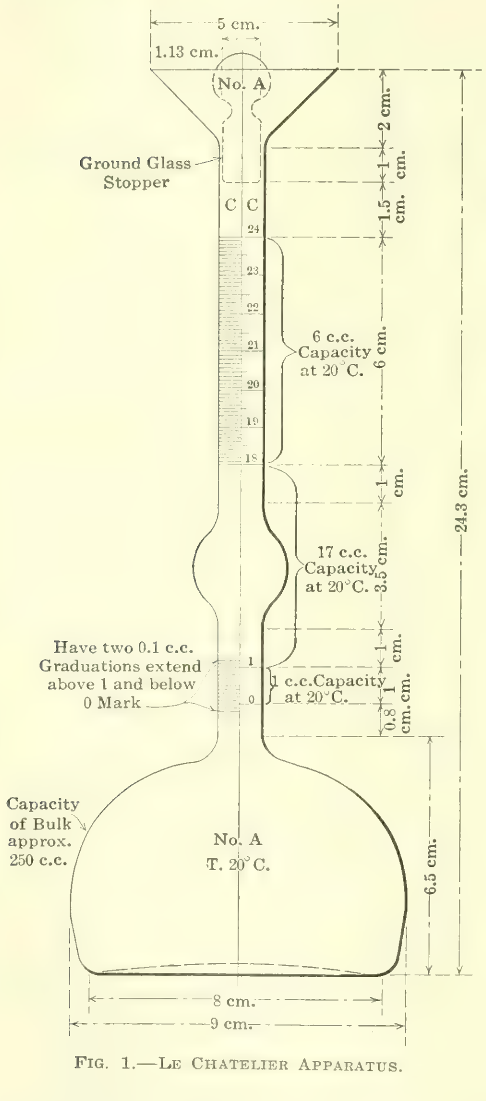


FIG. 1.—LE CHATELIER APPARATUS.

The sieve frames shall be circular, approximately 8 in. in diameter, and may be provided with a pan and cover.

33.—A standard No. 200 sieve is one having nominally an 0.0029-in. opening and 200 wires per inch standardized by the U. S. Bureau of Standards, and conforming to the following requirements:

The No. 200 sieve should have 200 wires per inch, and the number of wires in any whole inch shall not be outside the limits of 192 to 208. No opening between adjacent parallel wires shall be more than 0.0050 in. in width. The diameter of the wire should be 0.0021 in. and the average diameter shall not be outside the limits 0.0019 to 0.0023 in. The value of the sieve as determined by sieving tests made in conformity with the standard specification for these tests on a standardized cement which gives a residue of 25 to 20 per cent. on the No. 200 sieve, or on other similarly graded material, shall not show a variation of more than 1.5 per cent. above or below the standards maintained at the Bureau of Standards.

Method.

34.—The test shall be made with 50 g. of cement. The sieve shall be thoroughly clean and dry. The cement shall be placed on the No. 200 sieve, with pan and cover attached, if desired, and shall be held in one hand in a slightly inclined position so that the sample will be well distributed over the sieve, at the same time gently striking the side about 150 times per minute against the palm of the other hand on the up stroke. The sieve shall be turned every 25 strokes about one-sixth of a revolution in the same direction. The operation shall continue until not more than 0.05 g. passes through in one minute of continuous sieving. The fineness shall be determined from the weight of the residue on the sieve expressed as a percentage of the weight of the original sample.

35.—Mechanical sieving devices may be used, but the cement shall not be rejected if it meets the fineness requirement when tested by the hand method described in Section 34.

Permissible Variation.

36.—A permissible variation of 1 will be allowed, and all results in excess of the specified limit but within this permissible variation shall be reported as 22 per cent.

X.—Mixing Cement Pastes and Mortars.

Method.

37.—The quantity of dry material to be mixed at one time shall not exceed 1 000 g. nor be less than 500 g. The proportions of cement or cement and sand shall be stated by weight in grams of the dry materials; the quantity of water shall be expressed in cubic centimeters (1 cc. of water = 1 g.). The dry materials shall be weighed, placed upon a non-absorbent surface, thoroughly mixed dry if sand is used, and a crater formed in the center, into which the proper percentage of clean water shall be poured; the material on the outer edge shall be turned into the crater by the aid of a trowel. After an interval of

½ minute for the absorption of the water the operation shall be completed by continuous, vigorous mixing, squeezing and kneading with the hands for at least one minute.[1] During the operation of mixing, the hands should be protected by rubber gloves.

38.—The temperature of the room and the mixing water shall be maintained as nearly as practicable at 21° C. (70° F.).

XI.—Normal Consistency.

Apparatus. *39.*—The Vicat apparatus consists of a frame *A* (Fig. 2) bearing a movable rod *B*, weighing 300 g., one end *C* being 1 cm. in diameter for a distance of 6 cm., the other having a removable needle *D*, 1 mm. in diameter, 6 cm. long. The rod is reversible, and can be held in

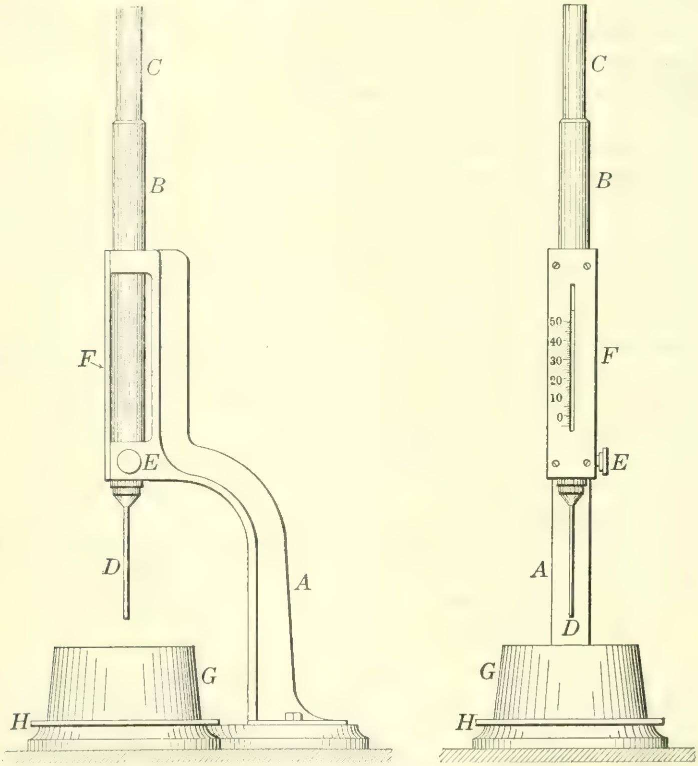


Fig. 2.—Vicat Apparatus.

[1] In order to secure uniformity in the results of tests for the time of setting and tensile strength the manner of mixing above described should be carefully followed. At least one minute is necessary to obtain the desired plasticity which is not appreciably affected by continuing the mixing for several minutes. The exact time necessary is dependent upon the personal equation of the operator. The error in mixing should be on the side of over mixing.

any desired position by a screw *E*, and has midway between the ends a mark *F* which moves under a scale (graduated to millimeters) attached to the frame *A*. The paste is held in a conical, hard-rubber ring *G*, 7 cm. in diameter at the base, 4 cm. high, resting on a glass plate *H* about 10 cm. square.

40.—In making the determination, 500 g. of cement, with a measured quantity of water, shall be kneaded into a paste, as described in Section 37, and quickly formed into a ball with the hands, completing the operation by tossing it six times from one hand to the other, maintained about 6 in. apart; the ball resting in the palm of one hand shall be pressed into the larger end of the rubber ring held in the other hand, completely filling the ring with paste; the excess at the larger end shall then be removed by a single movement of the palm of the hand; the ring shall then be placed on its larger end on a glass plate and the excess paste at the smaller end sliced off at the top of the ring by a single oblique stroke of a trowel held at a slight angle with the top of the ring. During these operations care shall be taken not to compress the paste. The paste confined in the ring, resting on the plate, shall be placed under the rod, the larger end of which shall be brought in contact with the surface of the paste; the scale shall be then read, and the rod quickly released. The paste shall be of normal consistency when the rod settles to a point 10 mm. below the original surface in ½ minute after being released. The apparatus shall be free from all vibrations during the test. Trial pastes shall be made with varying percentages of water until the normal consistency is obtained. The amount of water required shall be expressed in percentage by weight of the dry cement. Method.

41.—The consistency of standard mortar shall depend on the amount of water required to produce a paste of normal consistency from the same sample of cement. Having determined the normal consistency of the sample, the consistency of standard mortar made from the same sample shall be as indicated in Table 1, the values being in percentage of the combined dry weights of the cement and standard sand.

TABLE 1.—PERCENTAGE OF WATER FOR STANDARD MORTARS.

Percentage of water for neat cement paste of normal consistency.	Percentage of water for one cement, three standard Ottawa sand.	Percentage of water for neat cement paste of normal consistency.	Percentage of water for one cement, three standard Ottawa sand.
15	9.0	23	10.3
16	9.2	24	10.5
17	9.3	25	10.7
18	9.5	26	10.8
19	9.7	27	11.0
20	9.8	28	11.2
21	10.0	29	11.3
22	10.2	30	11.5

XII.—Determination of Soundness.[1]

Apparatus.

42.—A steam apparatus, which can be maintained at a temperature between 98 and 100° C., or one similar to that shown in Fig. 3, is recommended. The capacity of this apparatus may be increased by using a rack for holding the pats in a vertical or inclined position.

Method.

43.—A pat from cement paste of normal consistency about 3 in. in diameter, ½ in. thick at the center, and tapering to a thin edge, shall be made on clean glass plates about 4 in. square, and stored in moist air for 24 hours. In molding the pat, the cement paste shall first be flattened on the glass and the pat then formed by drawing the trowel from the outer edge toward the center.

44.—The pat shall then be placed in an atmosphere of steam at a temperature between 98 and 100° C. upon a suitable support 1 in. above boiling water for 5 hours.

45.—Should the pat leave the plate, distortion may be detected best with a straight edge applied to the surface which was in contact with the plate.

XIII.—Determination of Time of Setting.

46.—The following are alternate methods, either of which may be used as ordered:

Vicat Apparatus.

47.—The time of setting shall be determined with the Vicat apparatus described in Section 39. (See Fig. 2.)

Vicat Method.

48.—A paste of normal consistency shall be molded in the hard-rubber ring *G* as described in Section 40, and placed under the rod *B*, the smaller end of which shall then be carefully brought in contact with the surface of the paste, and the rod quickly released. The initial set shall be said to have occurred when the needle ceases to pass a point 5 mm. above the glass plate in ½ minute after being released; and the final set, when the needle does not sink visibly into the paste. The test pieces shall be kept in moist air during the test. This may be accomplished by placing them on a rack over water contained in a pan and covered by a damp cloth, kept from contact with them by means of a wire screen; or they may be stored in a moist closet. Care shall be taken to keep the needle clean, as the collection of cement on the sides of the needle retards the penetration, while cement on the point may increase the penetration. The time of setting is affected not only by the percentage and temperature of the water used and the amount of kneading the paste receives, but by the temperature and humidity of the air, and its determination is therefore only approximate.

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[1] Unsoundness is usually manifested by change in volume which causes distortion, cracking, checking or disintegration.

Pats improperly made or exposed to drying may develop what are known as shrinkage cracks within the first 24 hours and are not an indication of unsoundness. These conditions are illustrated in Fig. 4.

The failure of the pats to remain on the glass or the cracking of the glass to which the pats are attached does not necessarily indicate unsoundness.

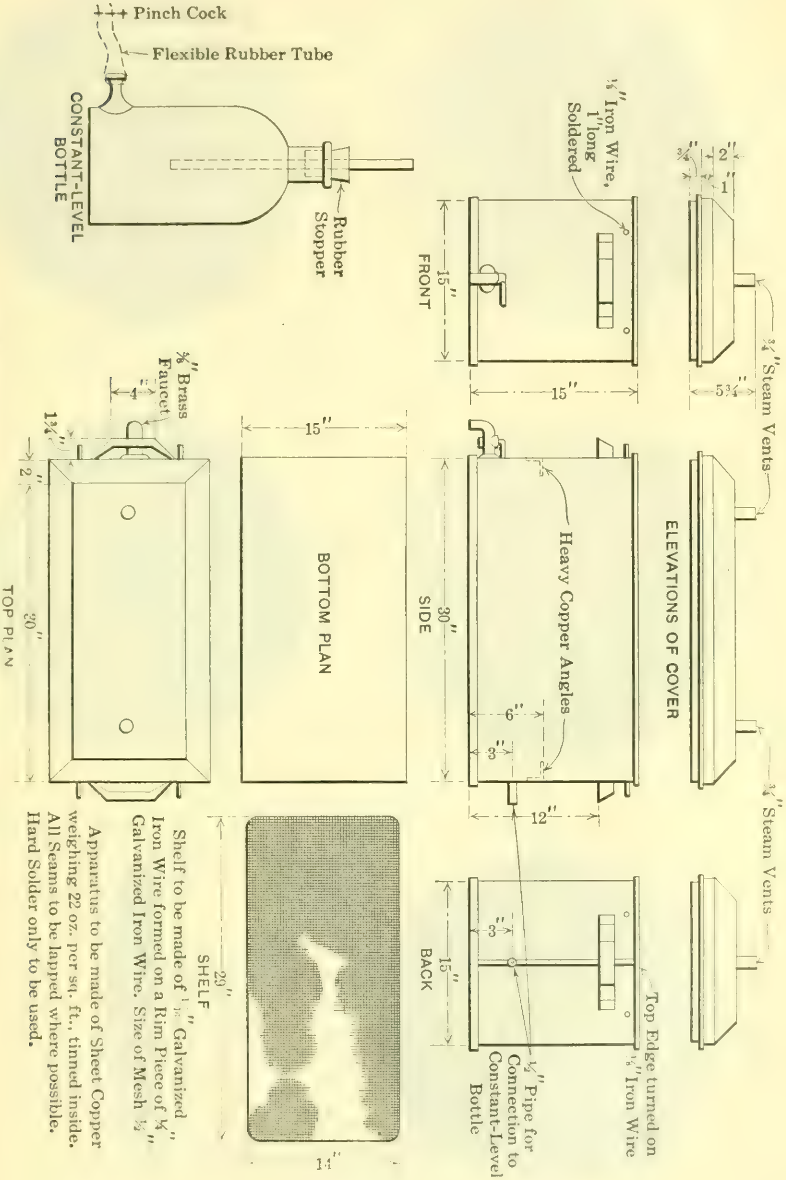


FIG. 3.—APPARATUS FOR MAKING SOUNDNESS TEST OF CEMENT.

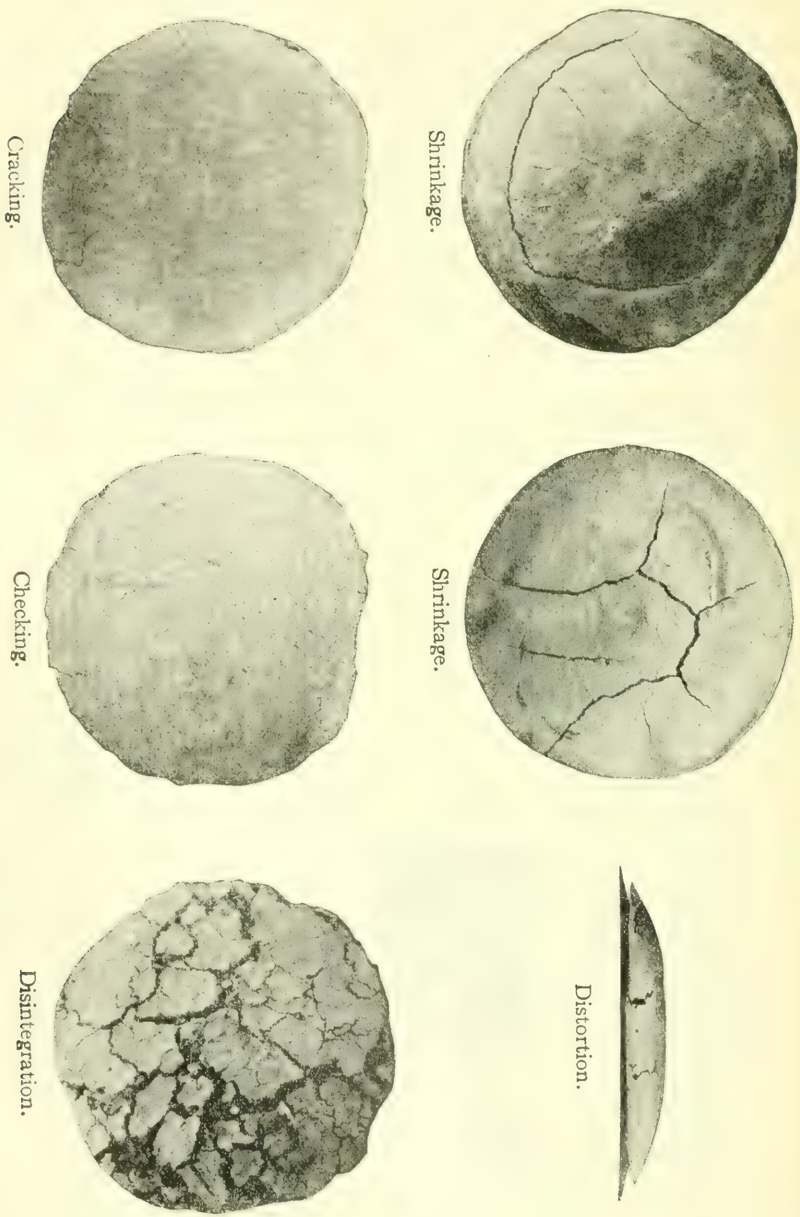
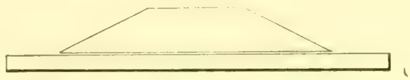


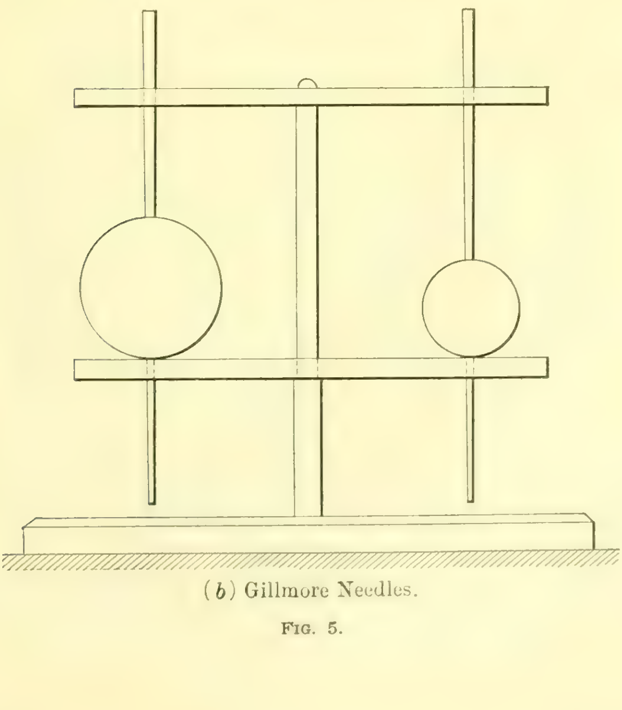
Fig. 4.—Typical Failures in Soundness Test.

49.—The time of setting shall be determined by the Gillmore needles. The Gillmore needles should preferably be mounted as shown in Fig. 5 (*b*). Gillmore Needles.

50.—The time of setting shall be determined as follows: A pat of neat cement paste about 3 in. in diameter and ½ in. in thickness with a flat top (Fig. 5 (*a*)), mixed to a normal consistency, shall be kept in moist air at a temperature maintained as nearly as practicable at 21° C. (70° F.). The cement shall be considered to have acquired its initial set when the pat will bear, without appreciable indentation, the Gillmore needle $\frac{1}{12}$ in. in diameter, loaded to weigh ¼ lb. The final set has been acquired when the pat will bear without appreciable indentation, the Gillmore needle $\frac{1}{24}$ in. in diameter, loaded to weigh 1 lb. In making the test, the needles shall be held in a vertical position, and applied lightly to the surface of the pat. Gillmore Method.



(*a*) Pat with Top Surface Flattened for Determining Time of Setting by Gillmore Method.



(*b*) Gillmore Needles.

FIG. 5.

XIV.—Tension Tests.

Form of Test Piece.

51.—The form of test piece shown in Fig. 6 shall be used. The molds shall be made of non-corroding metal and have sufficient material in the sides to prevent spreading during molding. Gang molds when used shall be of the type shown in Fig. 7. Molds shall be wiped with an oily cloth before using.

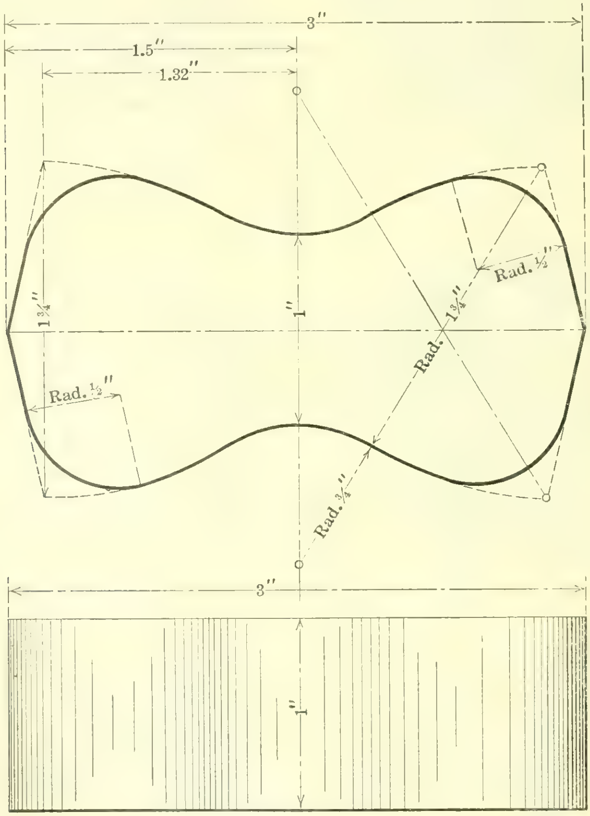


Fig. 6.—Details for Briquette.

Standard Sand.

52.—The sand to be used shall be natural sand from Ottawa, Ill., screened to pass a No. 20 sieve and retained on a No. 30 sieve. This sand may be obtained from the Ottawa Silica Co., at a cost of two cents per pound, f. o. b. cars, Ottawa, Ill.

53.—This sand, having passed the No. 20 sieve, shall be considered standard when not more than 5 g. pass the No. 30 sieve after one minute continuous sieving of a 500-g. sample.

54.—The sieves shall conform to the following specifications:

The No. 20 sieve shall have between 19.5 and 20.5 wires per whole inch of the warp wires and between 19 and 21 wires per whole inch of the shoot wires. The diameter of the wire should be 0.0165 in. and the average diameter shall not be outside the limits of 0.0160 and 0.0170 in.

The No. 30 sieve shall have between 29.5 and 30.5 wires per whole inch of the warp wires and between 28.5 and 31.5 wires per whole inch of the shoot wires. The diameter of the wire should be 0.0110 in. and the average diameter shall not be outside the limits 0.0105 to 0.0115 in.

55.—Immediately after mixing, the standard mortar shall be placed in the molds, pressed in firmly with the thumbs and smoothed off with a trowel without ramming. Additional mortar shall be heaped above the mold and smoothed off with a trowel; the trowel shall be drawn over the mold in such a manner as to exert a moderate pressure on the material. The mold shall then be turned over and the operation of heaping, thumbing and smoothing off repeated. Molding.

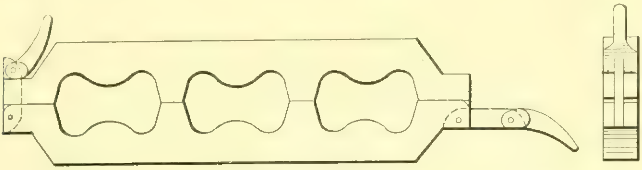


FIG. 7.—GANG MOLD.

56.—Tests shall be made with any standard machine. The briquettes shall be tested as soon as they are removed from the water. The bearing surfaces of the clips and briquettes shall be free from grains of sand or dirt. The briquettes shall be carefully centered and the load applied continuously at the rate of 600 lb. per minute. Testing.

57.—Testing machines should be frequently calibrated in order to determine their accuracy.

58.—Briquettes that are manifestly faulty, or which give strengths differing more than 15 per cent. from the average value of all test pieces made from the same sample and broken at the same period, shall not be considered in determining the tensile strength. Faulty Briquettes.

XV.—STORAGE OF TEST PIECES.

59.—The moist closet may consist of a soapstone, slate or concrete box, or a wooden box lined with metal. If a wooden box is used, the interior should be covered with felt or broad wicking kept wet. The bottom of the moist closet should be covered with water. The interior of the closet should be provided with non-absorbent shelves on which to place the test pieces, the shelves being so arranged that they may be withdrawn readily. Apparatus.

Methods.

60.—Unless otherwise specified all test pieces, immediately after molding, shall be placed in the moist closet for from 20 to 24 hours.

61.—The briquettes shall be kept in molds on glass plates in the moist closet for at least 20 hours. After 24 hours in moist air the briquettes shall be immersed in clean water in storage tanks of non-corroding material.

62.—The air and water shall be maintained as nearly as practicable at a temperature of 21° C. (70° F.).

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

OBSTRUCTION OF BRIDGE PIERS TO THE FLOW OF WATER

Discussion.*

By Charles Evan Fowler, M. Am. Soc. C. E.

Charles Evan Fowler,† M. Am. Soc. C. E.—The subject of the proper form of bridge piers has been discussed ever since modern bridge engineering has been practiced. The speaker has covered the practical part of this subject quite fully in his book, "Sub-Aqueous Foundations", in which the essential features are shown by Figs. 324 and 325. Mr. Fowler.

The construction of piers in a stream modifies the usual currents and flow of the water to such an extent as to endanger very often the foundations themselves. Early French writers thought to have solved completely the problem as to the form of a pier to cause the least disturbance, and M. Bossut, from a mathematical investigation, concluded that the starling should be a triangle, the nose being a right angle. M. Dubaut gave another and more nearly correct solution in his "Principles of Hydraulics", the faces of the starling to be convex curves tangent to the faces of the pier.

Experiments recorded in Cresy's "History of Civil Engineering" are partly illustrated in Fig. 21. The models were 15 cm. in thickness and the velocity of the current was 3.09 m. per sec. The illustrations show the direction and height of the induced currents and eddies, and the relative value of the various forms of the pier cross-sections, the best practical form being the section Fig. 21 (*e*), where the starling is formed of two convex curves tangent to the sides

* Discussion on the paper by Floyd A. Nagler, Jun. Am. Soc. C. E., continued from September, 1917, *Proceedings*.

† New York City.

Mr. Fowler.

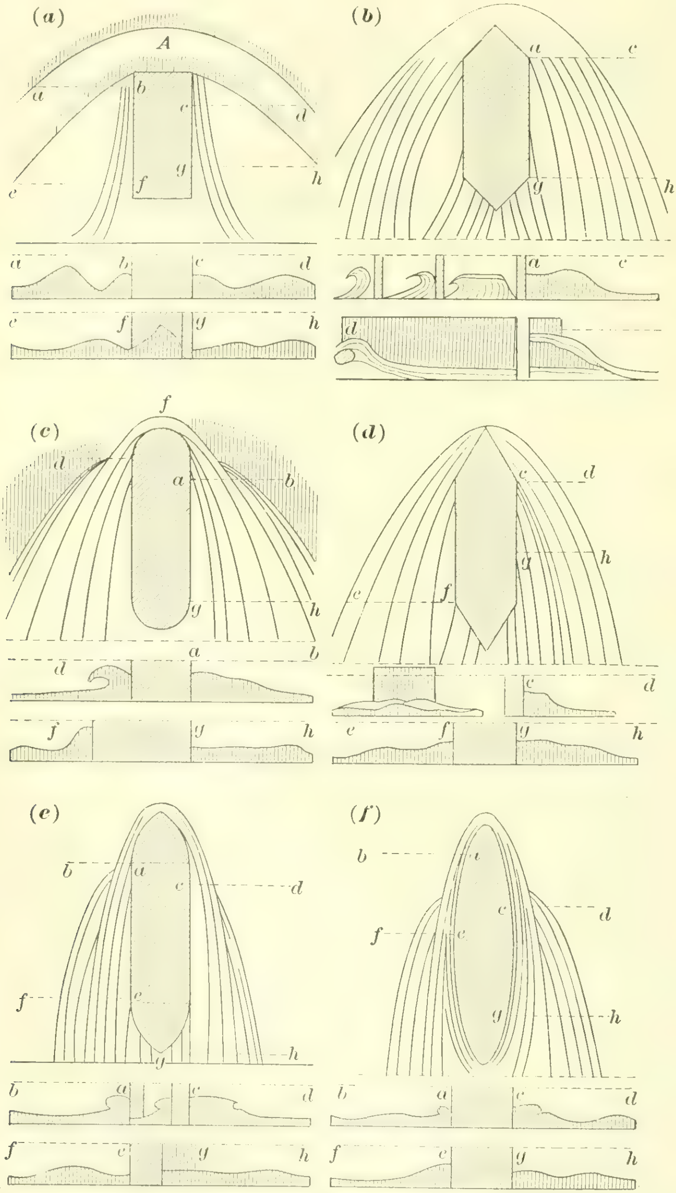


FIG. 21.—CRESY'S EXPERIMENTS ON THE FORM OF PIERS.

and described on an equilateral triangle. The very best form, however, is the elliptical, Fig. 21 (*f*). Mr. Fowler.

The piers of the Knoxville steel arched cantilever were of the cross-section shown in Fig. 21 (*e*), and were equally spaced by the speaker, thus making the structure symmetrical, and contributing to its architectural appearance.

The late George S. Morison, Past-President, Am. Soc. C. E., also adopted this form for most of the piers built by him in the Missouri and Mississippi Rivers, but, from a point just above high water, he used semicircular ends. This was a very good design, both for practical and esthetic purposes.

Gustav Lindenthal, M. Am. Soc. C. E., stated to the speaker recently, that, in his opinion, long spans would only be possible during the next 20 to 30 years, owing to a future normal advance in the price of steel, due to the very rapid depletion of the raw materials for steel manufacture.

Although the form of piers is vital with long spans, as regards mainly the integrity of the piers themselves, the subject will become of increasing importance as engineers are forced more and more to build short-span structures of reinforced concrete. This type is already being constructed in numerous locations where a thorough investigation of the best form of pier is necessary.

The speaker, however, does not believe that the results obtained from the experiments under discussion are of more than tentative value, owing to the conditions of the tests being wholly unlike those encountered in actual practice, where the materials and methods of pier construction become vital factors of the problem. Then, there are the portions of the cribs below water, rip-rap protection around piers, the character of the river bottom, the scouring actions induced, and many other items, to which consideration must be given. Therefore, it would seem that, until we have the results from a series of experiments, conducted on lines more nearly in accordance with actual conditions, with several piers and on a larger scale, we can only accept those general conclusions which are in reality confirmatory of the findings of the data from Cresy.

The form then, as shown in Fig. 21 (*e*), as adopted by Mr. Morison, and as used by the speaker at Knoxville and elsewhere, would seem to be the logical one to adopt ordinarily, using care at the same time to avoid unnecessary obstruction from cribs, rip-rap, or other protection work.

Although it has been largely out of the question to adopt the better form, or elliptical cross-section, as shown in Fig. 21 (*f*), where piers have been constructed of stone, there is no reason at all for not adopting such a form for concrete piers, where the obstruction factor becomes

Mr. Fowler. very vital. The Fowler patent reinforced concrete pier has come to be widely used on account of its great saving in material and consequent great saving in weight on the foundation bed. This pier, consisting of two reinforced concrete pillars connected by a web of similar construction, undoubtedly offers a much greater obstruction than the usual forms of solid masonry, and experiments in the future should include this and any similar or new types.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

THE THREE 15-CUBIC YARD DIPPER-DREDGES, GAMBOA, PARAISO, AND CASCADAS, AS SUPPLIED AND USED ON THE PANAMA CANAL

Discussion.*

By Messrs. Arthur W. Manton, A. W. Robinson, and William M. Rosewater.

Mr. Manton.

Arthur W. Manton,† M. Am. Soc. C. E. (by letter).‡—There are a few points relative to these dredges about which the writer would like to have more information, although, perhaps, it may not be considered within the scope of the paper.

The writer spent some time on the dredges at the Pacific end of the Panama Canal, with James Macfarlane, Superintendent of Dredging, and was much interested in the performance of the Lobnitz bucket-and-ladder dredges built in Scotland for the French Company. The work done by these old dredges, although in softer material, was considered to be excellent, and, as regards cost, very satisfactory. Since then, the writer has also had the opportunity of studying the work performed by the bucket-and-ladder dredges on the Suez Canal.

After noting the very considerable and expensive repairs and renewals (materials and labor) on the three dredges, as stated in the paper, the writer cannot help thinking that perhaps the advantages—from some "angles"—of the bucket-and-ladder type of dredge may have been overlooked in adopting the dipper-dredge design; and it is hoped that the author will give the reasons for the adoption of the latter. Apparently, the stresses on the boom, dipper handle, spuds, and hull of the three dredges were enormous; and, in order to deal with large masses of rock, it appears that something had to be sacrificed.

* Discussion of the paper by Ray W. Berdeau, Jun. Am. Soc. C. E., continued from October, 1917, *Proceedings*.

† New York City.

‡ Received by the Secretary, October 3d, 1917.

Mr. ...nton. The bucket-and-ladder dredges which the writer has operated abroad had large outputs, with small repairs and maintenance charges, both as to labor and material, throughout extended periods of work, although they were subject to severe stresses in a seaway, due to bad weather; in one case, especially, at Portsmouth Dock Yard, one of the dredges was moored and working in the seaway, and had to be hauled out of its "cut" very rapidly in order to permit battleships to pass in and out of the yard. No spuds were provided, and, in this respect, American practice seems to be excellent. If the dredge had been provided with a rear swinging spud, the work would have been improved. The channel bottom had to be dredged to a depth which required an accuracy limit of about 6 in. The material was compact gravel, although not specially hard to dig, and the face varied from 3 to about 12 ft. The buckets had a capacity of about 27 cu. ft., and were operated at about 11 per min., the dredge capacity—a barge-loading dredge—being between 500 and 600 cu. yd., *in situ,* per hour.

Under the circumstances the cost figures were distinctly reasonable, although they would have been considerably less had the dredge been able to operate for the full 24 hours, instead of at low tide—daylight hours only—as the Admiralty would not permit night dredging in the fairway.

Referring again to the cost of the three "dippers", the writer would be glad if the repairs-and-maintenance cost (divided into materials and labor) could be separated from the actual operation cost, and if the costs of fuel and wire rope could be given separately; it is inferred that the latter was very high. He would also like to know what was provided for depreciation. It is assumed that the staff superintendence cost, insurance cost, and general charges are not included in the figures given; the operation time-losses per 24 hours for each cause would also be interesting.

The maintenance cost (nearly 150% of the operation cost), appears to be high, and this would lead one to suppose (although this point was not brought out so fully by discussion perhaps as it might have been) that the dredge was operated in a way not at all anticipated; or that the rapid completion of the Gaillard Cut was considered to be so important that it was a small matter to run the dredge in such a way as to distress it in a manner not provided for by the manufacturers, or, indeed, the engineers. This point was only brought out by Mr. Rosewater, but from the paper the dredges may be said to have worked exceedingly well under the onerous conditions imposed.

The use of a bucket-and-ladder dredge, with buckets of, say, 40 cu. ft. capacity, with possibly one "horn", in hard ground, to every two ladder-buckets, doubtless would have involved the breaking up of the larger boulders, say, from 10 to 20 tons, subsequent to the passing

Mr. Manton.

of the dredge; but this might be considered as an advantage, for it would keep the dredge at work for a greater proportion of the 24 hours. Mr. Rosewater has stated that boulders were brought up—sometimes twenty times daily—so large as to stick in the bucket, or to necessitate blasting them in the scow to enable them to pass through the doors. Dealing with large boulders with such a machine as a dipper-dredge would seem to delay unduly the dredge, scows, and tugs, and, in addition, to increase seriously the repairs and maintenance costs.

With the bucket-and-ladder dredge, the smaller buckets would work around such boulders, ultimately tumbling them into the bottom of the cut, to be lifted subsequently with a crane, or blown up to be re-dredged.

There may be some novel points in this discussion, which may elicit a reply as to the reasons for making these dredges of this particular design. They were provided with buckets of two sizes; if 15 cu. yd. was the best size for soft excavation, would it not appear that the 10-cu. yd. bucket for hard and soft rock would be a somewhat large, and therefore expensive, size, involving these excessive stresses and undue operating delays?

Mr. Robinson.

A. W. Robinson,* M. Am. Soc. C. E. (by letter).†—The dipper-dredges described by Mr. Berdeau are probably the most important examples of this type yet used on large work. The dipper-dredge is especially suited to canals and narrow channels, because it takes up little room, and does not require anchorage lines which would obstruct navigation, as in the case of the ladder dredge, nor does it move its hull at all when working. These dredges were the main reliance in reducing the slides which involved enormous volumes of excavation during the first years of traffic on the Panama Canal. The work done shows the great capability of this type, each of the three dredges having a demonstrated capacity of more than 3 000 000 cu. yd. per year.

The work of the third machine—ordered after some experience gained with the first two—naturally shows improvement of detail, which is reflected in the better monthly average output. It is to be regretted that further and more radical improvements were not made in order to reduce the abnormally high cost of maintenance which shows in all three examples, which, if eliminated, would have permitted still larger outputs, and at much less cost.

The cost of maintenance, normally, should be only a small fraction of the cost of operation, but the figures given show that the "maintenance" of the *Gamboa* was 1.5 times the cost of operation, that of the *Paraiso* being 1.7 times, and that of the *Cascadas* 1.4 times. "Maintenance" is stated in the paper to be the "cost of keeping the equipment

* Montreal, Que., Canada.

† Received by the Secretary, October 26th, 1917.

Mr. Robinson. in first-class condition, and depreciation only." In other words, repairs and depreciation.

The cost of "maintenance" of the *Gamboa* was $131 350 for 1915, and that of the *Paraiso* was $125 000. The *Cascadas* was no better, but rather worse, despite the improvement in design, being $108 111 for 9 months of 1916, or at the rate of $144 000 for the year. These are heavy figures, and, after making all due allowance for hard driving and conditions of work, they indicate the existence of serious defects of design. As intimated in the paper, the principal defects are to be found in the main hoisting ropes, the dipper arms, and spuds.

The life of the main hoisting rope is stated to be from 3 to 35 days. The replacement of a steel rope, 3¼ in. in diameter and 275 ft. long, at such frequent intervals is a heavy tax, to say nothing of the loss by delay. The first dipper-dredge to be fitted with direct wire rope hoist was built by the Bucyrus Company in 1890, and of this the dredges under discussion are the lineal descendants.

As compared with the old 3-part chain hoist then in use the direct hoist with single rope was an instant success, and has been used widely. For modern heavy loads, however, the single rope is quite unsuitable, and the writer has developed a system of multiple ropes, consisting of two or more ropes in parallel, by which all the advantages of speed and simplicity of the direct single rope are retained, with great durability.

The importance of the subject of wire rope hoisting for heavy loads at speed merits further remark.

It is commonly supposed that, for good results, the sheave diameter should bear a certain ratio to the rope diameter, say 30 diameters or more. A leading maker of wire ropes lists the minimum size of drum and sheaves in a table which for a 6-strand, 19-wire, cast-steel rope, 1 in. in diameter, is 4 ft., and for a rope 2 in. in diameter is 8 ft. It is quite erroneous to proportion sheave diameters to rope diameters in direct ratio. According to this, assuming that a rope 1 in. in diameter would work well over a sheave 30 in. in diameter (which is true in practice if the rope is flexible and the load and speed are moderate), a rope 3¼ in. in diameter would require a sheave 30 by 3¼ in., or, say, 8 ft., which is what we find in the dredges under discussion, and is manifestly too small. It is clear that the resistance to bending of a member is not a linear function of the depth. In solid round members the moment of inertia varies as the fourth power of the depth or thickness. In a wire rope in which the individual wires compose the section, the problem is more complex, owing to friction between the wires under varying tension. In practice, the writer finds that the resistance to bending is proportional to the square of the diameter of the rope, and that sheave diameters proportioned accordingly give good results. Thus, if a rope 1 in. in diameter requires a 30-in. sheave, a 2-in. rope should have a sheave 22 by 30, or 120 in. in diameter, and a 3¼-in. rope would

Mr. Robinson.

require a sheave about 26 ft. in diameter, instead of 8 ft. as used. Not only is the sheave diameter entirely too small, but the rope in question is subject to eight bendings per lift in passing over the two sheaves, or twelve bendings over three sheaves, not counting the wind in the drum. This is equal to from 700 to 1 000 bendings per hour. It is also subjected to severe twisting on the vertical axis as the boom revolves, and, as this twisting and untwisting takes place in the short distance of 13 ft., it is most injurious to a rope of this size. It is hardly necessary to say that such a rope, used in this way, violates mechanical principles, and must necessarily be short lived, no matter what kind of "extra flexible" or what grade of steel is used.

Another common error is to assume the same diameter for drum and sheaves, as in the maker's catalogue referred to. Drums may be considerably smaller than sheaves, for equal wear. This is evident when we consider that the rope makes two bendings in passing over a sheave, once on and once off, and only one bending in winding on a drum. The wear from bending, therefore, is only half as much on a drum as on a sheave of equal size.

The dipper arm used on these dredges is of the split type, and about 40 tons in weight. The use of the split type of arm with rack and pinion is a survival from early and small dipper-dredges for light and shallow work, and, in the writer's opinion, is quite unsuitable when applied to modern heavy dredges for deep water. The concentration of load due to the weight alone on the narrow shroudings of the rack is excessive, and the load and wear on the teeth, when pinched, due to side motion and general stress and twisting, is quite beyond the safe limit of endurance. Furthermore, the lateral strength and resistance to twisting of the split type of arm are small, and the construction is such that, when over-stressed, permanent deformation takes place. Consequently, the wear and tear is severe, and the cost and delay due to the renewal of these parts is a large item. No amount of increase of section of this design will remedy the trouble; on the contrary, it will aggravate it. The great weight of the dipper arm is a predisposing cause of wear, and likewise is very severe on the dipper when dropped on the bottom.

The question of spuds for heavy stresses in deep water is an important one, and here, again, the methods of early days fail, and an entirely new treatment of the subject is demanded. For many years, and for depths, on the Great Lakes, a plain wooden spud sliding in plain wooden vertical guides answered every purpose. As depths and stresses increased, steel spuds were tried, but failed in many cases owing to lack of resilience and also to the concentration of the load at one point. The ordinary methods of construction become inadequate and unsuitable when we have to deal with the enormous stresses generated by machines of great power working in hard material

Mr. Robinson.

and in deep water. The pressure of a square spud against the sides of a parallel slide in which it fits more or less loosely is nil along the middle portion of the slide and is concentrated at the top and bottom; this results in shearing the rivets at points of concentration, and often causes deformation and rupture. Furthermore, a steel spud being a rigid member in a more or less rigid casing, there is no elasticity, as in a wooden spud. Strengthening the parts and making the spud of heavier section does not remedy the trouble, on the contrary, with an enormously heavy spud, the difficulty of handling it is increased, and the rivets loosen up as before, unless suitable methods are adopted to distribute the stress over a sufficient number of rivets and provide the necessary elasticity.

The author states that these spuds have to be removed every 90 days, three having been broken in as many days in one case.

The stresses to which they are subject cannot be closely calculated, as in a bridge, but no bridge designer would think of subjecting a riveted box-beam, of the dimensions stated, to a bending stress of 150 tons or more concentrated on a few rivets, such a beam also bearing, as a column, a heavy vertical load.

In addition to the foregoing points of criticism, to which the author has drawn attention, the following additional points may be mentioned.

Method of Stiffening Hull.—The steel hull is stiffened by two fore-and-aft steel trusses. This method is a survival from the days of wooden hulls, which required stiffening. The writer thinks that better stiffness, with less weight and obstruction of space, can be provided by a suitably designed steel hull, without trusses, but which possesses the necessary strength in itself. The bottom of the hull, the steel main deck, and the top member of the truss, together constitute a beam with three flanges, which is an absurdity. Some excuse for the truss is found in the support it gives to the overhead turn-table, but this would be much better on deck, where it would be on a solid foundation, and would permit the operator to have an unobstructed view of his work, a defect to which the paper calls attention.

Main Engines.—The main engines are compound, with outside cranks. The writer thinks that simple engines, with center cranks, would be preferable. Compounding an engine which stops every 40 sec., runs at full load about 10% of the time, and at all sorts of speeds, does not result in any economy. In locomotives, which run more uniformly, compounding has gone out of use principally on account of the value of simplicity and good starting power—both essentials on a dipper-dredge—and because the economy was not sufficiently marked to make it worth while. Some economy might be had by adopting higher pressures and by superheating. The side cranks, again, are a relic from small hoisting engines used in early days,

and are out of place in marine practice, or in large engines subject to sudden changes of speed and load. Mr. Robinson.

Boom.—The boom is of the curved bowstring design, which the writer used for many years, but found deficient in lateral stiffness against the side thrust of the dipper arm, and weak at the lower end, due to its shallow depth at that place and its inability to resist twisting stresses. Therefore, he designed a boom with straight tapering members, having wide bottom flanges and great depth at the lower end; this provided freedom of movement to roll, instead of attempting to resist rigidly the stresses due to swinging. This boom is sometimes known as the "Atlantic type"; it possesses great strength, with less weight and fewer parts.

Inaccessibility of Parts.—The author has already commented on the difficulty of dismounting the brake wheels and stripper shaft, due to design. The writer would also criticize the arrangement of having so much steam machinery below deck in a tropical climate, some of it, such as the spud engines behind the trusses, in an inaccessible position.

Capacity of Dipper.—Though called 15-cu. yd. dredges, it does not appear that they have ever been worked with a dipper of that size, even in soft material, and 10 cu. yd. seems to be the useful limit of capacity, although the dimensions of the dipper, as stated in the paper (10½ by 9 by 9 ft. over all), would indicate a somewhat less capacity. With a very large dipper, it is not alone a question of the power and ability of the dredge to handle it, but of the ability of the scow to receive the impact of such a mass of material and dispose of it.

Character of Work.—The paper states that these dredges were "placed in Gaillard Cut in rock digging exclusively." A more specific classification of material dredged would be necessary in order to form a just idea of the work done. Although some rock may have been present, "rock digging exclusively" is misleading. The sliding in of the banks of the canal, and the large output, both prohibit the idea of "rock exclusively."

The defects which have been criticized illustrate a tendency to follow beaten tracks, which lead astray when followed too far. This tendency is found in many branches of engineering, but especially in the design of dredging machinery, the early development of which has been largely in the hands of so-called practical men with slight acquaintance with engineering. By a process of strengthening parts that failed or proved weak, successive examples were built up into a system from which it was thought unwise or impossible to depart. In many instances, tons of metal were introduced into the design of parts in this way as the result of experience, and the degree of success resulting was an apparent justification, whereas the truth is that entirely different designs based on engineering principles and sound reasoning would have been better.

Mr. Robinson. The splendid work which these dredges have done in spite of the mechanical defects noted is a tribute not only to the excellence and solidity of the machines as a whole, but especially to good operation and management at a time when the passage of the Canal was at stake.

Mr. Rosewater. WILLIAM M. ROSEWATER,* ASSOC. M. AM. SOC. C. E.—This paper gives a brief description of the machinery used on these dredges, together with some suggestions for changes which, in the author's opinion, would improve future dredges. The performances of these dredges are tabulated, and some figures are given relating to the output and cost of operation, but, unfortunately, the conditions of operation are not described, and, without knowing these conditions, the records of performances cannot be intelligently compared or used in predicting what other dredges, or even the same dredge, may do on other work.

The speaker will try to describe the conditions under which these performances were made, and also explain how the original specifications for these dredges were decided on.

At the time the first large slide closed the Canal to navigation, the Panama Canal Commission was operating a number of hydraulic dredges, a few dipper-dredges equipped with 5-yd. dippers, and the ladder-dredge, *Corozal,* with buckets of about 35 cu. ft. capacity. All these dredges were found to be unsuitable for clearing away this tremendous mass of broken rock and earth. The dipper-dredges were too small and of too light construction, and the *Corozal* was unsuitable for the work because the material had lost all its original regularity of formation, and was no longer in layers of original uniform material, through which one could steadily feed an endless chain of buckets, but was a mass of earth mixed with broken pieces of rock of all sizes and shapes, and lacking all uniformity of formation, a mass against which, on account of this lack of uniformity, no endless chain of buckets, with its uniform speed and original close formation (as compared with the enormous size of the pieces of rock), could be expected to cope successfully.

Such a mass calls rather for a large single bucket or dipper, the direction, point of application, and speed of which are constantly under control, and with which the operator can feel his way as he plows through the mass.

Such were the conditions that led to the placing of orders for these large dipper-dredges. A few years previous to this, W. G. Comber, M. Am. Soc. C. E., then in charge of the dredging, had visited the United States to inspect the more important large dipper-dredges on the Atlantic Coast and the Great Lakes. Of all these dredges only two types were considered as coming within the requirements for this work; these were the 15-yd. dipper-dredge, *Toledo,* built by the Bucyrus Company

* South Milwaukee, Wis.

for George H. Breymann and Brothers, of Boston, Mass., and the two 10-yd. dipper-dredges working on the Cape Cod Canal. The latter were sister machines, and were built by the Atlantic Equipment Company, frequently referred to as the American Locomotive Company. After careful inspection and study of these dredges, the officials of the Panama Canal decided that the *Toledo* represented the type most suitable for their work. The *Gamboa* and the *Paraiso* were, with certain modifications, to be duplicates of the *Toledo,* the largest dredge of this type, and practically the only one that met, in a general way, all the requirements of the work on the Canal. This dredge carries a 12 or 15-cu. yd. dipper, depending on the nature of the material to be dredged, and can dig to a depth of 50 ft. below the water line. Mr. Rosewater.

The *Toledo* has a wooden hull, with structural steel stiffening trusses, a single boiler, and wooden spuds, the forward spuds being operated by a two-part wire rope tackle. The Panama dredges were to have steel hulls, also structural steel trusses, two boilers, and steel spuds, the forward spuds to be operated by a four-part wire rope tackle. In practically all other respects these dredges were to be duplicates of the *Toledo.* Suggestions for improvements in the design of the main hoisting engine, drums, and gears had been made by the speaker's company at that time, but were rejected by the Canal officials, chiefly on account of the extra time it would have taken to get out an entirely new design. The need for these dredges on the Canal was so great that, in view of the satisfactory work done by the *Toledo,* the Canal officials felt justified in rejecting suggestions for changes in design which if incorporated would have delayed the delivery of the dredges.

When the order for the third dredge, the *Cascadas,* was placed, the first two had been in operation for about 8 months. The work of these dredges had been highly satisfactory, and the only changes requested were that the hull be widened so that it would be flush with the outside of the forward spuds. This increased the width of the hull about 11 ft., and added to its transverse stability in maneuvering the dredge with the spuds in the high position. It also provided space for a third boiler, which was considered desirable. A gallows frame was also to be added at the bow, so as to permit raising the forward spuds from their tops, thereby avoiding the use of sheaves at the bottom of the spuds, as in that position they were easily clogged with mud, and were a source of more or less trouble on the older dredges. Changes in the design of the main hoisting engine, drums, and gears were again considered, and rejected as not being worth while, but it was decided to omit the reversing links and add a separate reversible engine, to be thrown into gear, when required, to turn the main drum slowly and thus facilitate the renewal of the cable and when making repairs.

Mr. Rosewater.

The performance of all these dredges is unique, and the speaker doubts very much that it can ever be equalled elsewhere, even with the same or similar dredges, because all the conditions necessary for continuous maximum output will probably never again be present to the same extent as on the Canal at the time these performances were made.

The conditions necessary for maximum output are as follows:

1.—A depth of cut sufficient to insure a full dipper for every stroke.

2.—A channel free from traffic, thus causing no delays on account of passing vessels.

3.—A channel suitable to permit setting adrift the loaded scows as fast as filled. These were pushed out of the way by the empty scow, and picked up later by a steam tug. This method of handling scows could not be used at all times, of course, because some of the slides completely closed the Canal, and then, naturally, the loaded scow had to be moved out of the way before an empty one could take its place, and, at times, the scows even had to be turned end for end before they could be completely loaded.

4.—No delays caused by waiting for empty scows. This required, not only an ample supply of scows and steam tugs, and a perfect transportation system, but also a dumping place free from rough weather and storms, so as not to interrupt the towing and dumping service.

5.—A bank of earth slowly moving toward the dredge, so that the latter could work for long periods of time without having to stop to move ahead.

6.—Careful and expert management, with experienced crews, and the division of the 24-hour working day into three 8-hour shifts, and not into two 12-hour shifts.

From Tables 1, 2, and 3, the minimum and maximum monthly outputs for these dredges are shown to be:

In 1914	for the *Gamboa*	108 000 cu. yd.	to 166 000 cu. yd.
and	" " *Paraiso*	70 000 " "	to 144 000 " "
In 1915	" " *Gamboa*	150 000 " "	to 280 000 " "
and	" " *Paraiso*	134 000 " "	to 313 000 " "
In 1916	" " *Gamboa*	129 000 " "	to 321 000 " "
and	" " *Paraiso*	196 000 " "	to 306 000 " "
and	" " *Cascadas*	122 000 " "	to 330 000 " "

If the output, as given in these tables, is averaged for the fiscal years, the *Gamboa* and *Paraiso,* for the first year's operation, each removed 150 000 cu. yd. per month, and for the second year's operation 250 000 cu. yd. per month for each dredge; the average for the *Cascadas* during its 8 months' work in 1916 was 300 000 cu. yd. per month.

The *Gamboa* and *Paraiso*, at the start, were operated on only two 8-hour shifts per day, and were changed to three 8-hour shifts at about the time the *Cascadas* was placed in commission. The *Cascadas* was always operated on three 8-hour shifts. The increase of 100 000 cu. yd. per month for the second year's work of the *Gamboa* and *Paraiso* is no doubt largely due to the addition of the third shift, which added about 50% more operating time to each month, but the scow service was also improved at this same time, so that it is the combination of these two that really accounts for the increased yardage. Mr. Rosewater.

Assuming twenty-six 24-hour working days per month, the average hourly output for both the *Gamboa* and *Paraiso* was 400 cu. yd., and for the *Cascadas* about 480 cu. yd. The *Gamboa* and *Paraiso* were originally equipped with 10-yd. and 15-yd. dippers, for work in rock and earth, respectively, but it was found that the rock and earth were mixed to such an extent that no distinguishing line could be drawn dividing the classes of work, nor was it found practicable to change the dippers with the change in the character of the material, so that these dredges used the 10-yd. dippers constantly, and the 15-yd. dippers were eventually cut down and converted to rock dippers. The *Cascadas* was equipped with a 12-yd. dipper, suitable for both rock and earth work, and a similar change was intended to be made on the other dredges, but the speaker thinks that this had not been done at the time of these performances. Assuming, then, that the *Gamboa* and *Paraiso* were using 10-yd. dippers, and the *Cascadas* a 12-yd. dipper, each of the three dredges was loading at an average of not less than forty dippers an hour, or a dipperful every 1½ min. To make this average output, the dredges had to work very much faster than this, for these figures do not allow for lost operating time; and, to illustrate just how fast it was possible to operate, *The Canal Record* of February 23d, 1916, stated that on February 18th, 1916, in a 24-hour period, from midnight to midnight, the *Cascadas* loaded into scows 23 305 cu. yd. of earth and rock in 23 hours and 15 min. actual working time. This is at the rate of 1 002 cu. yd. per hour, or 16.7 cu. yd. per min., or a dipperful about every 40 sec. This also illustrates the fact that the dredge lost practically no time whatever in doing this work, and that it probably was not even required to change its position, as a shift of position usually causes a delay of from a few minutes to an hour, depending on the location and the character of the ground in which the dredge is working.

In this same month, the shortest of the year, this dredge appears also to have made its record maximum output of 330 000 cu. yd., an average of 550 cu. yd. per hour, which is approximately 55% of its best single day's record.

Mr. Rosewater. The best record for the *Toledo,* so far as the speaker knows, has been 9 000 cu. yd. in 15 hours, and her monthly output varies from 100 000 to 200 000 cu. yd., but the conditions under which most of her work was done are widely different from those existing at the Panama Canal. The depth of cut generally was shallow, from 6 to 10 ft., and the time lost in moving the dredge ahead was generally very appreciable, but the largest loss of time is usually caused by rough seas, which prevent taking the scows out to the dump. At some seasons of the year this loss amounts to practically two-thirds of the available working time.

From the foregoing it will be readily seen to what an extent the conditions at Panama favor the work and affect the yardage obtained. For efficient results, not only must the dredge be of proper strength and design, and suitable for the work, but the transportation problem, also, must be very carefully solved, in order to take full advantage of the possibilities of the dredge.

Tables 1, 2, and 3 also give the cost of operation per cubic yard, and show that it was remarkably low. These figures, however, cover only the cost of loading the material into scows, and do not include the towing and dumping. As dredging contracts, as a rule, include towing and dumping, the speaker would suggest that Mr. Berdeau add, if possible, a supplement showing the cost per cubic yard of spoil on the dump, including all overhead charges, and also giving the length of tow per round trip. From figures pertaining to this work which the speaker has seen, the cost of towing will add approximately 20 cents per cu. yd., and will show that the cost of spoil on the dump varies from 24 to 33 cents per cu. yd. These figures cover the performance of the *Gamboa* and *Paraiso* from January to October, 1915. The length of the tow per round trip was about 25 miles. For the *Cascadas,* on account of its slightly larger average output, the cost figures ought to be slightly less than those given.

Some of the details will now be taken up in the order in which they appear in the paper. Mr. Berdeau submitted to the speaker's company a copy of his paper for correction before it was printed, but it is found that several slight errors were apparently overlooked, and, for the sake of accuracy, the speaker will try to correct them.

On page 946,* the acceptance of the *Cascadas* is given at Port Richmond, N. Y. This is an error. The *Gamboa* and *Paraiso* were completed and accepted at Port Richmond, N. Y., but the hull of the *Cascadas* was built by the New York Shipbuilding Company at Camden, N. J., where the Bucyrus Company put in the machinery and completed the dredge.

The next correction is in regard to the depth of the *Gamboa* and *Paraiso.* The hulls of these dredges are 16 ft. 6 in. deep at the bow and 13 ft. 6 in. at the stern.

* *Proceedings,* Am. Soc. C. E., August, 1917.

Mr. Rosewater.

The dippers or buckets, as supplied by the Bucyrus Company, were of stronger and heavier design than any that had ever been built before. Each 10-yd. rock dipper weighed nearly 20 tons, and each 15-yd. dipper, 21 tons. The rock dippers had been in service only a comparatively short time when they began to show signs of distress, and had to be overhauled. An attempt was made to hold the Bucyrus Company responsible for the failure of these dippers, on the ground that this failure showed they were too light and inadequate for the work. The speaker's company, however, had no difficulty in presenting proof to F. C. Boggs, M. Am. Soc. C. E., Major, Corps of Engineers, U. S. A., then Purchasing Officer for the Panama Canal, that these dippers were stronger and heavier than any then in service, and that the company had faithfully and to the best of its ability fulfilled its contract.

These dippers are attached to a handle weighing approximately 37½ tons, and, together with the dipper itself, there is a total weight of 57½ tons in motion, and if, in lowering, the operator misjudges the distance, he can easily allow the dipper to strike bottom with sufficient momentum to be crushed. The speaker's company quite naturally concluded, from the first meager reports cabled to it, that this had been the cause of the trouble, just as Mr. Berdeau has done in his paper. However, the detailed reports, when received, showed that the parts had been forced outward and not inward, as would have been the case if the destructive force at work had been due to the weight of these parts; and, finally, when all the facts became known, it developed that these dippers frequently brought up large pieces of rock, too large to go through the dipper and also too large to pass through the openings in the dump scows. Sometimes these pieces were lodged on top of the dipper and at other times they were wedged tightly in place, and as they were too large to pass through the dump scows, they had to be broken up. This was done by placing a light charge of dynamite on top of the rock, covering it with clay, and then lowering the dipper below the water surface, but still allowing it to hang on the hoisting rope when the charge was exploded. This happened sometimes as often as twenty times in a working day, and it not only broke up the rock, but also eventually the dipper. This naturally had not been contemplated in designing these dippers, and it fully accounted for the manner in which the parts failed. It is not at all surprising, therefore, that these dippers had to be overhauled and repaired every 3 or 4 weeks. This treatment of the rock was also responsible for some of the short-lived hoisting cables. Inspection of the cables showed that blasting the rock in the dipper broke a few wires of the cable each time, and naturally shortened its life.

Mr. Rosewater.

Cables.—The life of the cable in ordinary service is usually represented by 300 000 cu. yd. The life of the cables at Panama, in view of the continuous high-speed operation of the dredges, and the treatment that was sometimes given these cables, does not seem excessively short.

The suggestion has been made that sheaves of larger diameter would increase the life of the cables, but the speaker doubts that this would result in any material benefit, because the drum diameters are considerably less than those of the sheaves, and a cable usually gives way at the point where the radius of bend is the smallest.

Dipper Handles.—It was specified that the dipper handles should be duplicates of the one on the *Toledo,* but more heavily armored; the handle of the *Toledo* had originally been specified to be a duplicate of one used on an older and smaller dredge, so as to make one spare handle serve for either dredge. In this way the width of 12 in. for each dipper stick was determined arbitrarily, and was incorporated in the design of the Panama dredges. The speaker has always been of the opinion that a stick 16 in. wide would be far better for dredges of this size, but a change in the width of these sticks would entail many other changes which would affect the initial cost of the dredge quite materially, so that, in the end, the adoption or rejection of the suggested improvement usually rests with the purchaser, and not with the Engineering Department.

Saddle Blocks.—The saddle blocks, as stated by Mr. Berdeau, are of the latest and most approved type, and have given general satisfaction. The block is on the outside of the handle sticks, instead of between them, and is provided with adjustable caps for holding the handle rack in proper mesh with its pinion. It is essential to keep the saddle block in proper adjustment as the handle wears, for the forces at work tend to separate the rack from its pinion, and, if the clearance is allowed to become too great, this will cause the teeth to jam and break, and even break the saddle block itself. Rollers in place of slide-plates would undoubtedly reduce the wear at this point, and Mr. Berdeau's suggestion for their use is a good one, but, before it can be adopted, the old and long established prejudice against rollers for this service will have to be overcome. Some 15 or 20 years ago, rollers were considered standard practice, but the design was inadequate, and caused these roller slide-plates to be condemned.

The use of the single dipper stick with a divided hoisting rope—one rope on each side of the stick—has been suggested from time to time. The difficulty with this is that no one has yet found a way to lead the ropes from the widely separated sheaves at the foot of the boom through the turn-table to the sheaves in the hold at the bow of the dredge, and still retain the proper lead of the rope

for all positions of the boom. Nor has any arrangement yet been perfected that would permit leading these cables directly from the point of the boom to the drum without going through the turntable. Such an arrangement is often used on Lidgerwood hoists operating clam-shell buckets, but cannot be used on a dipper-dredge because of the interference of the dipper handle with these cables as the boom swings toward the corner of the dredge. Mr. Rosewater.

On some of the large revolving shovels built by the Bucyrus Company the two members of the dipper handle have been separated far enough to allow the entire boom to be placed between them. This permits the use of a box-girder boom having maximum strength and stiffness for any given weight of material. It also increases the torsional resistance of the handle on account of the wide spread of its members, but this design is not well adapted for use on deep-digging dipper-dredges because of the interference in certain positions of the boom of the handle with the A-frame legs. This might be overcome by using A-frames with bent or bowed legs, but such frames would have to be of extremely heavy construction in order to take care of the added bending moment. The widely separated dipper sticks would also require a broad section of dead wood below the boom, which would add greatly to the resistance in pulling the dipper back with the backing cable, especially if working in a strong current; and a further objection might be raised to this that the current would tend to break the dipper sticks by forcing them against the bottom corner of the hull.

Booms.—The booms are alike on all three dredges, and are of plate-girder construction, being 25% heavier than the one on the *Toledo,* which is of the latticed-girder type. The boom, without doubt, is the most difficult part of the dredge to design. It is subject to extremely heavy loads, some of which are indeterminate; its shape and the location of some of its principal members are very often fixed by the physical requirements, so that the result is at best more or less of a compromise between what one would like to do and what one can actually do. By indeterminate loads the speaker means those caused by starting to swing the boom before the dipper is clear of the water, and sometimes before it is clear of the bank, also those caused by the sudden stoppage and reversal of swing necessary when working at the highest speed. Then, too, it is not uncommon practice to move the dredge sidewise by swinging the boom with the dipper bedded on the bottom. All these forces produce reversal of stresses, and have a far-reaching effect that sooner or later makes it necessary to overhaul the boom, no matter how well it was designed and built. It has been the speaker's experience that the heavier and stronger the boom is built, the more likely it is to be overworked; in other words, when the boom

Mr. Rosewater. looks extra strong and heavy, the operators do not hesitate to do with it what they would be afraid to undertake with some of the lighter booms that are in successful operation.

Main Engines.—The main engines are of the twin, tandem, compound type. In a 40 or 50-sec. cycle of operation, these engines run only for 10, 12, or 15 sec., at most, and are idle for the remainder of the time. From this, the speaker believes, it will be readily perceived that the conditions can hardly be right for the use of a compound engine, and for this reason the speaker has always advised against their use, but, as these dredges show, without success. The compound engines were insisted upon for the *Toledo,* and thus they came to be used on the Panama dredges.

The speaker is frank to confess that, as far as he knows, no indicator cards have ever been taken from any of these compound engines, so that he does not know what the low-pressure cylinder is actually doing. Mr. Macfarlane, the Superintendent of Dredging on the Canal, stated a year ago that, if he were to order a fourth dredge, he would want it provided with simple, double-cylinder engines, and not compound.

The statement in the paper that the low-pressure cylinders have piston valves is in error. It is the high-pressure cylinders that have the piston valves, the low-pressure cylinders have double-ported slide-valves, with American balanced slide-valve discs.

Hoisting Drum and Gear.—The drum is of the so-called differential type, that is, it is of two diameters, the connection between the small and large diameter being made by two or three turns of a grooved spiral section. The original idea of this, as explained in the paper, was to obtain a slow speed and maximum digging effort when the dipper was on the bottom and a fast speed with proportionately reduced effort while the loaded dipper was being hoisted.

The small part of the drum is usually of a diameter that is rather small for the size of the cable, and its length, proportioned for average conditions, cannot possibly be right for all conditions. Thus, on a deep-digging dredge, unless the cable is especially shortened to suit the job in hand, most of the work in a shallow cut will be done on the spiral and large diameter parts of the drum, and the dredge will thus be at a disadvantage for power. This would also be true for a deep cut in any soil in which the dipper must cut its way clear to the surface of the water. Trouble has also been experienced with drums of this type on the older dredges, which are now working with reduced boiler pressure, in the event that the operator misjudges his clearance height and stops hoisting before the dipper is high enough to pass the hatch combing of the scow, for then, if he is swinging fast, he cannot stop, nor can he get the hoisting engine to start the loaded dipper from rest in

that position without first lowering it to obtain a more favorable starting position, and, when there is not time for this, a collision and damage to both scow and dredge is inevitable. Such accidents, however, do not happen where the original boiler pressure is carried, for the speaker always aims to have sufficient hoisting power to start the dipper from any position. Mr. Rosewater.

To overcome these objections, of late years, the speaker has been recommending the straight drum of large diameter, and with gears properly proportioned to give the correct digging effort and speed. With a drum of this type the available power will not be affected materially by the depth of digging, and the engine can be readily designed so as to accelerate rapidly and run at higher speed the moment the dipper finishes its cut and the comparatively easy work of hoisting the load begins.

One of the chief obstacles to the straight drum of large diameter has been its increased initial cost. The larger drum calls for more powerful friction clutches and brakes, and requires also heavier and more powerful gearing. This, translated into commercial terms, means increased initial cost, and usually resolves itself into a question that the purchaser decides without much reference to the engineering points involved.

The use of twin cables running side by side has been advocated by many, and tried on a number of large dredges, as a solution of the hoisting cable question; in fact, twin cables were suggested for use on the *Cascadas.* It was rejected by the Government engineers, after careful consideration. The chief argument against the use of the twin cables is that it is impossible to maintain them at exactly the same length. They will stretch differently, and there will be slight differences in the off lead from the sheaves to the drum which will also tend to vary their length, so that an efficient equalizing arrangement is necessary, and, with the machinery as arranged in most dredges, the only place for this equalizer is at the dipper bail, where the space available will not permit the use of sheaves of large diameter; with sheaves of small diameter, the life of the twin cables has been no greater than that of the single cable, and the cost of the twin cables has been greater.

Swinging Circle and its Machinery.—The swinging circle and its machinery are of the usual type constructed by the Bucyrus Company. The division of each swinging rope into two parts, to facilitate its renewal, and to conserve incidentally the portion that receives very little wear, is so simple an improvement that it seems remarkable that no one had ever thought of it before. The reason for this, no doubt, is that ordinarily these cables are changed so infrequently and that there is usually plenty of spare time to make this change while the

Mr. Rosewater. dredge is lying idle, that no one ever thought of making an improvement here until the conditions changed as they did at Panama.

The mechanism for controlling the throttle-valve and reversing the swinging engines with a single lever is new. On the older and smaller dredges this was effected by merely connecting the links and the throttle to the same lever system, but, on the larger dredges, the links require too much power to permit them to be operated in this manner. On these dredges, therefore, the links are reversed by a steam cylinder the direction of motion of which is controlled by a special slide-valve. The arrangement used connects this controlling slide-valve with the lever system controlling the throttle-valve in such a way that, starting from the central or neutral position of the operating handle, the movement of the handle in either direction causes the links, during the first portion of the lever travel, to be placed in correct position corresponding to the direction in which the handle has been moved, and further movement of the handle then no longer affects the position of the links, but serves only to regulate the throttle-valve. This arrangement has been highly successful, and provides a safe and practically fool-proof method for controlling both the direction and the speed of swinging by using a single hand lever. The only error the operator can make is to move his lever the wrong way thereby starting the boom to swing in the opposite direction from that intended, but, as the lever requires no power for its operation, he can usually discover his error and correct it before any serious damage is done. A similar mechanism is also used for controlling the direction and speed of travel of the forward spuds.

Backing Engine.—Mr. Berdeau states that the diameter of the backing drum is too small for its rope. This has been the case in practically all dredges, and is caused by the fact that, originally, a chain was used for this purpose, and was still retained even after the hoisting, swinging, and spud handling chains were discarded for wire cables. The substitution of a cable for the backing chain, therefore, is only of comparatively recent date. When this change was made on the existing dredge, it was usually effected by salvaging the better part of a worn-out hoisting or swinging cable, and this could easily be done, as the backing cable is only 90 or 100 ft. long. This cable was then used on the drum which had previously wound the chain, and in this way the extremely small ratio of drum to cable diameters was established, and, as the results were comparatively satisfactory, especially where old pieces of other cables were used, there was no incentive to change the ratio, and, even on new dredges, this small ratio was retained when the purchaser discovered that the use of a larger drum, with its larger gears, clutches, and brakes, added a few hundred dollars to the initial cost of the machinery.

Forward Spud Machinery.—The chief improvement in the forward spud machinery consists in the use of four-part, in place of two-part, blocks, for handling the spuds. The necessity for this was brought about by the demand in recent years for dredges having booms which could swing through a total of 180 degrees. On the older dredges, the maximum swing seldom exceeded 120°, and with the increase to 180° in the arc of swing it was soon discovered that greater loads were placed on the forward spuds, so that, to permit the dredge to dig with the boom swung to its limit, four parts of cable were required to carry the load. Mr. Rosewater.

On the older dredges, using only two parts of cable on the forward spuds, it was a comparatively simple matter to renew the cable, or even to change spuds, so that no one felt warranted in incurring the additional expense of providing a gallows frame for handling these spuds from their tops. When the four-part spud hoists were tried out on the Panama dredges, however, it was soon discovered that as the reaving of a new rope and the changing of spuds was very slow and difficult work, the use of an overhead gallows frame would be well worth its additional cost, and, for this reason, it was adopted on the *Cascadas*.

Mr. Berdeau points out that they are experiencing some trouble with the stern spud machinery because of the wear of the saddle-block parts that hold this spud in proper relation to the pinion with which it is hoisted and lowered. This trouble no doubt could be overcome by improving the design of the saddle block and substituting rollers for the slide-plates. If wire ropes were used for handling this spud, the service of a saddle block could be dispensed with, but wire ropes have not proved satisfactory for this purpose on deep-digging dredges. The chief objection to their use has been that they work so smoothly and noiselessly that the operator allows the spud to fall altogether too fast, and when he attempts to check its speed it frequently has gained enough momentum to break the rope or to tear out the connections for the rope, and, when this happens, the services of a diver are necessary to make repairs. Where the rack and pinion are used for handling this spud, the work is not done so smoothly and quietly, the result being that the noise usually serves to prevent the operator from letting this spud travel too fast.

Deck Fittings.—The two three-drum winches used for moving scows are a very important part of the deck equipment, and it was through their use that the dredges were supplied with a substantially continuous string of scows. In addition to these deck winches, there were also added two steam capstans, near the stern of the dredge, which were used for bringing empty scows up alongside and within reach of the lines from the other winch. These capstans were also useful when it was necessary to turn the scows end for end, in order to load both ends of the scow.

Mr. Rosewater.

Boiler and Fittings.—Special care was taken in the design of the steam piping and the fittings, so as to have everything as nearly perfect and complete as possible. The boilers burn crude oil, using the Dahl patent mechanical oil burners, which burn the oil under pressure, so that no steam is required for atomizing it.

House and Cabin Construction.—Everything in connection with the house and cabins was given a great deal of thought and attention, and all plans were forwarded to the officials at Panama for approval, so as to insure their serviceability for the climatic conditions on the Canal, and also to meet correctly the requirements for the "gold" and "silver" crew quarters.

Hull.—The decision to widen the hull for the *Cascadas* was brought about by the fact that, in maneuvering the other dredges, it was essential to keep the boom pointing straight ahead, especially if all the spuds had been raised high in the air. The operating levers are properly labeled to show what each controls, but, even so, in the haste with which work is being done, there is always the possibility that the operator may pull the wrong lever at the wrong time, and if, under such conditions, he should happen to swing the boom clear to the side, it might be possible to overturn the dredge. On the first two dredges the spuds are outside of the hull proper and project about 5½ ft. on each side. It was decided, therefore, to widen the hull by just this amount. This enabled the builders to provide space for the third boiler, and has resulted in a dredge that has sufficient transverse stability to withstand safely the swinging of the boom to the corner, even when the spuds are in their highest position. It may be added that this apparent lack of stability on the earlier dredges and also on the *Toledo* was well understood by the builders and the operators, and that it is customary to take the necessary precautions to prevent an accident of this kind. In the past, the objection to using a wider hull has been based on the size of the locks of the Welland Canal. These will not pass anything wider than 44 ft., and, consequently, for dredges built in America, there is usually a clause in the specifications to the effect that the dredge may be stripped down to this width.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

AIR TANKS ON PIPE LINES

Discussion.*

By Messrs. R. D. Johnson and P. Wahlman.

R. D. Johnson,† Esq. (by letter).‡—Mr. Warren has touched on the fact that such formulas for the simple air tank as those proposed by him have only a limited practical application, because they furnish no index of the nature of the surge wave, as to whether or not it will die out of its own accord after its inception. This is a general fault with formulas which neglect governor action. Mr. Johnson.

Where differential action is omitted, there is much likelihood of falling into the error of designing the tank too small, and inflicting on the power plant a pressure accumulator instead of providing it with a regulator, and thus causing far more trouble than that which is intended to be cured.

Therefore, the writer has thought it desirable to supplement his equations with the following analysis, which serves to fix a minimum limit to the size of the simple air tank, independent of any formula which purports to give the value of the first dip of the pressure wave, following a load change.

If a surge wave is an increasing one, no perceptible initial load change is required to quicken it into life, because the system is already in a state of unstable equilibrium. Therefore, if, by studying the effect of very small load changes, an accurate solution of the critical size of a tank can be obtained, there is perfect assurance that, if the wave once begins to augment, it will continue to do so indefinitely.

On the other hand, if the study of the infinitesimals shows a tendency of the wave to die out, it seems reasonable that all appreciable surges would show the same thing.

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* Discussion of the paper by Minton M. Warren, Assoc. M. Am. Soc. C. E., continued from October, 1917, *Proceedings*.

† New York City.

‡ Received by the Secretary, October 26th, 1917.

Mr. Johnson.

It is necessary to regard only very small load changes in applying this theory, because, in that case, certain quantities are negligible which otherwise would complicate the mathematics beyond the possibility of manipulation.

In the following, let Z be the total dip of the pressure wave, at the end of the first quarter cycle, from its initial quiescent level, or, in Mr. Warren's nomenclature, $Z = Y + p_1 - p_2$, and let z be the variable, Z.

The following well-known fundamental equations hold good when governor action and friction are neglected, and V_2 is regarded as a constant draft velocity of the water-wheel.

$$d\,t = \frac{\left(\frac{L}{g}\right) d\,v}{z}, \text{ and } d\,y = \frac{(V_2 - v)\,a\,d\,t}{A}.$$

Also, when Y is small compared to l, we may write,

$$d\,z = \frac{p_1 + l}{l}\,d\,y.$$

Eliminating dt, we have,

$$\int_0^Z z\,d\,z = \frac{a\,L\,(p_1 + l)}{A\,l\,g}\int_{V_1}^{V_2}(V_2 - v)\,dv.$$

From which,

$$Z = \sqrt{\frac{a\,L\,(p_1 + l)}{A\,l\,g}} = (V_2 - V_1) \ldots\ldots\ldots\ldots\ldots (1)$$

This expression holds good only for a small value of Z when V_2 is very little larger than V_1, the difference being called Δv.

The wave described by this formula would live everlastingly at its initial amplitude, that is, when not influenced either by the damping action of friction or the accelerating effect of governor action, and all dimensions of air tanks would fulfill the so-called critical condition, of a uniform, everlasting wave.

It follows, logically, that if the damping action of friction is just equal, in its effect, to the augmenting action of the governor, then the wave continues in the same condition as though neither influence was present.*

Therefore, if these two opposing factors are equated, the critical dimensions of the air tank become apparent.

The increment to Δv caused by the effort of the governor to maintain constant power as the pressure drops, is, naturally, $\frac{Z\,V_2}{h - Z}$, where h is

* See "The Surge-Tank in Water-Power Plants", *Transactions*, Am. Soc. Mech. Engrs., 1908, p. 840.

the net head corresponding to p_1; or, as Z is here considered insignificant as compared to h, the increment to Δv may be written Mr. Johnson.

$$\frac{Z\ V_2}{h} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2)$$

The corresponding increment to Z or Δz may be found, by reference to Equation (1), to be,

$$\Delta z = \sqrt{\frac{a\ L\ (p_1 + l)}{A\ l\ g}} \times \frac{Z\ V_2}{h} \quad \ldots\ldots\ldots\ldots\ldots\ldots (3)$$

Now, the opposing change in friction head, due to Δv, may be obtained from the differential expression, $c\ \left((V_2 + \Delta v)^2 - V_2{}^2\right)$, which becomes (neglecting $\overline{\Delta v}^2$ as an infinitesimal of the second order)

$$\Delta f = 2\ c\ V_2 \Delta v \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (4)$$

Or, substituting the value of Δv from Equation (1),

$$\Delta f = 2\ c\ V_2\ Z \sqrt{\frac{A\ l\ g}{a\ L\ (p_1 + l)}} \quad \ldots\ldots\ldots\ldots\ldots\ldots (5)$$

Now, when $\Delta z = \Delta f$, the condition of instability sought is realized.

Equating their values, as derived in Equations (3) and (5), we have, for the dimensions of the critical size of simple air tank,

$$\frac{A\ l}{p_1 + l} = \frac{a\ L}{2\ g\ c\ h} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (6)^*$$

If A or l is smaller, or p_1 larger, than in the combination which equals this expression, then the design becomes, theoretically, an impossible one, and practically, also, unless some existing condition, such as partial differential action, a sluggish governor, or additional friction of some sort, has been overlooked.

Equation (6) involves the idea of constant efficiency of water-wheel, or a horizontal line expressing the relation between power output and efficiency, and therefore holds good only at the crest point of the efficiency curve through an infinitesimal range of surging.

If the water-wheel is operating on the drooping side of the efficiency curve, the tank dimensions need to be still more ample to reach the critical value.

If the negative tangent of the efficiency-power curve $\left(\text{that is, } \frac{\Delta E}{\Delta P}\right)$, where E is the efficiency and P is the power, be called tan. α, the correct expression involving this new idea is,

$$\frac{A\ l}{p_1 + l} = \frac{a\ L\ \left(E + \frac{3}{2} P \ \text{an.}\ \alpha\right)}{2\ g\ c\ h\ E} \quad \ldots\ldots\ldots\ldots\ldots (7)$$

* When l becomes infinite, this equation is seen to agree with the Thoma formula for the open simple tank.

Mr. Johnson.

A numerical substitution in Equation (6) indicates that Mr. Warren's chosen example falls far below the critical size, and, in such cases, as he has elsewhere intimated, his value of p_2 cannot be said to constitute even an approximation to the true magnitude of the maximum or minimum air pressure.

It must not be understood that Equation (7) provides a sufficiently large tank for practical purposes, because it distinctly does not.

An everlasting wave is a constant invitation to trouble due to partial synchronous conditions between the period of load-changes and that of the pressure wave, and, ordinarily, a commercially practicable simple air tank would need to be much larger than indicated by Equation (7).

It follows, therefore, that even correct equations, impossible to derive, relating to the first quarter cycle of the wave in a simple air tank which is unmodified by adequate choking devices, would be of very doubtful practical value, because the cost of a commercially operative simple air tank is usually found, from Equation (7), to be prohibitive.

Mr. Wahlman.

P. Wahlman,* M. Am. Soc. C. E. (by letter).†—For several years the writer has been interested in surge-tank regulation, but, so far, has not found any reliable formulas for the simple tank (that is, without the differential arrangement), whether open or closed as an air tank. Consequently, he has taken interest in testing the practical value of Mr. Warren's new formulas with the help of the tedious but accurate process of arithmetic integration.

For this test the author's own tank (page 1024‡) has been selected, and, in the arithmetical work, the friction as well as the governor action has been considered, as neither can be neglected in practice. As the friction and entry loss in the 2 100-ft. conduit are not given, it has been necessary to assume a value of that combined loss, such as $F = 0.07\ V^2$. The static head at the wheels has been assumed to be 459 ft.

Fig. 2 shows graphically the result of a complete shut-down from full load, or from $V = 12.5$. In this case, the governor has no influence, of course, and only the friction makes this practical case different from the author's first case. The correct result is $Y = 20.64$ and $p_2 = 779$, as compared with the author's 20.0 and 764, or 20.8 and 783 for isothermal expansion; and, consequently, it may be assumed that Equations (A) and (B) are practically correct for this particular case.

When the load is thrown on, the governor action is added and cannot be disregarded. The result of full load thrown on is shown graphically in Fig. 3, and a glance at these curves shows that, due

* New York City.

† Received by the Secretary, October 26th, 1917.

‡ *Proceedings*, Am. Soc. C. E., August, 1917.

Mr. Wahlman.

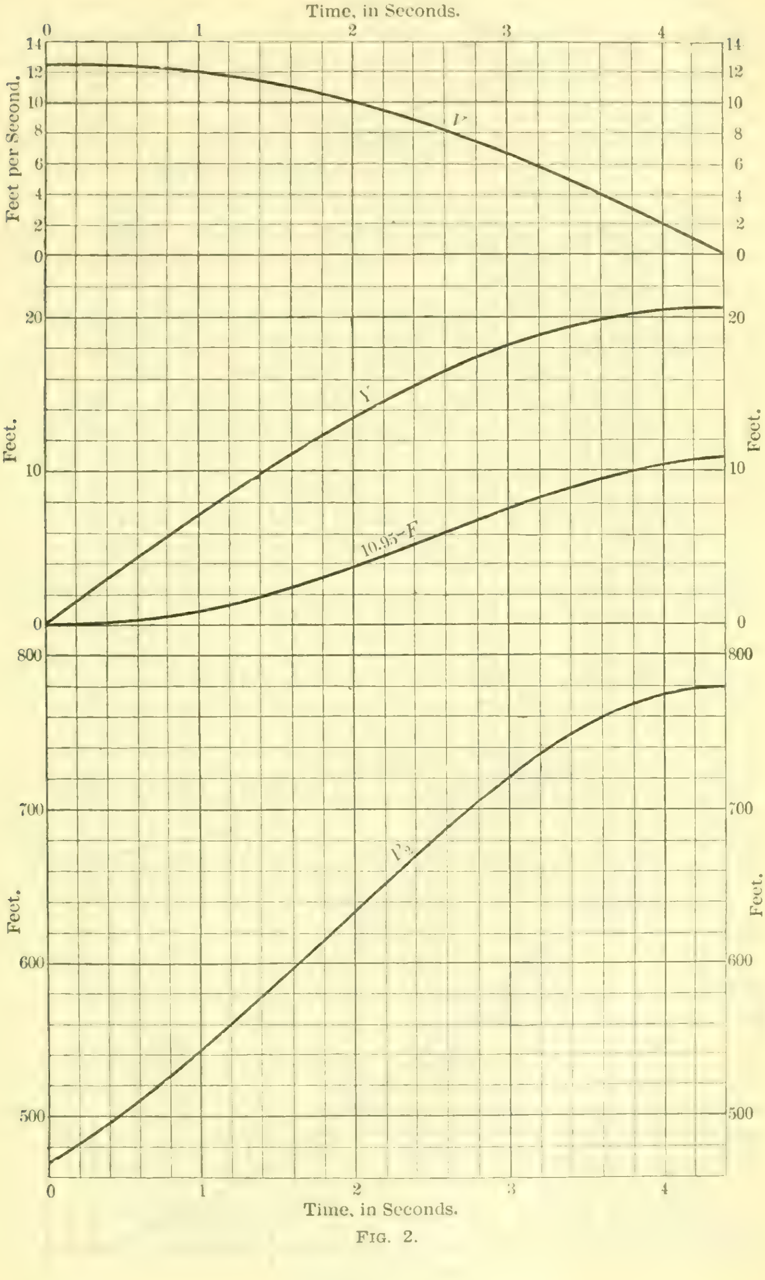

FIG. 2.

Mr. Wahlman.

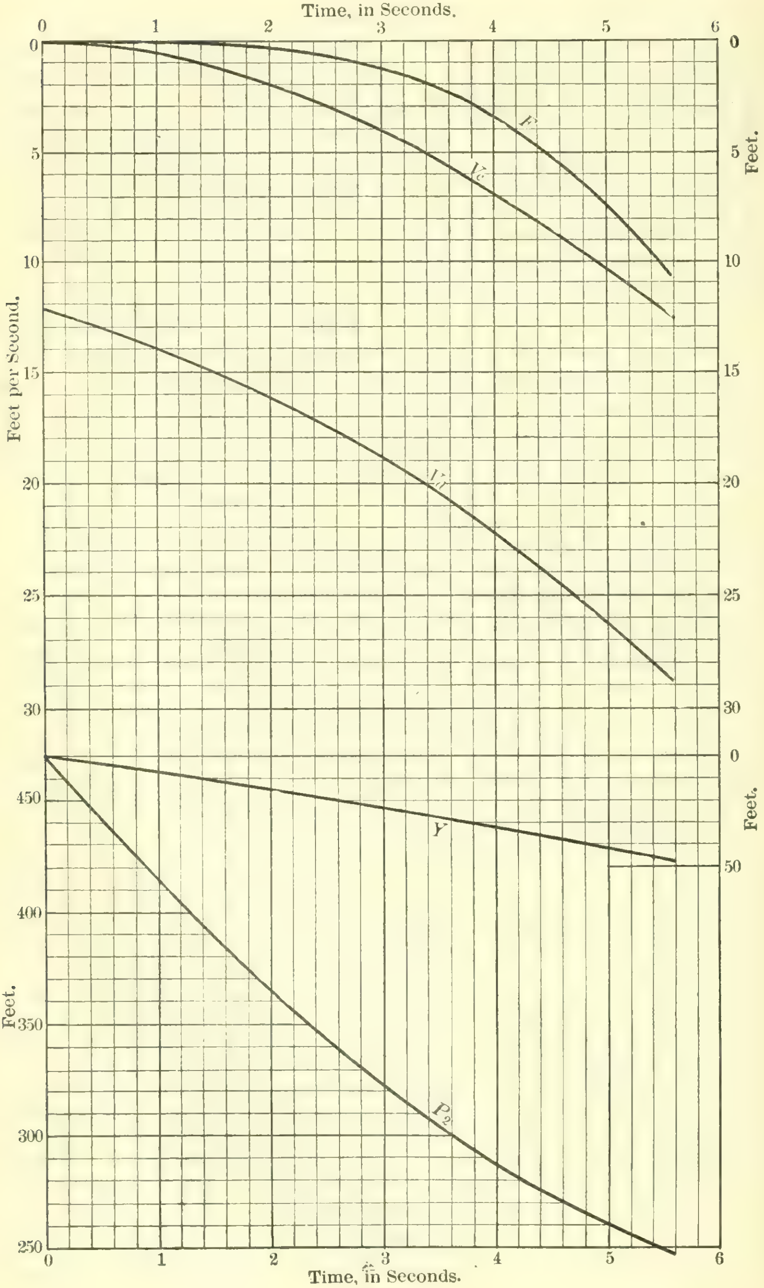

FIG. 3.

to the governor, the drop in head and the simultaneous increase in draft velocity, V_d, is so rapid, that the conduit velocity, V_c, cannot be accelerated fast enough to catch up with the draft velocity, which means that the full load cannot be maintained, even if the wheels were large enough to take the steadily increasing flow and their efficiency was constant. It is evident, therefore, that, for full load thrown on, the author's formulas are impractical. Mr. Wahlman.

Fig. 4 demonstrates the conditions when half load is thrown on suddenly. When the conduit velocity has first caught up with the draft velocity, Y is only 16.87, and $p_2 = 354$, and if the arithmetic integration was stopped at this point, one might be inclined to believe that the tank in question was sufficiently large for half load thrown on; but, if the curves are worked out farther, their amplitude is found to increase for each surge, and will finally go beyond possible limits; that is to say, the tank or conduit will either burst or the tank will be emptied.

Fig. 5 finally shows that even as small a sudden load change as 10% of full load will cause surges which will grow indefinitely. Consequently, it is evident that an air regulator of the dimensions indicated in the author's example is not only entirely inadequate for any sudden load changes, but even spoils regulation altogether, and that the formulas which the author has presented cannot be relied on for practical work.

Only isothermal expansion has been considered in the arithmetical work, as it is evident that the use of adiabatic expansion would have about the same effect as a smaller tank and constant temperature.

If this regulator is in practical use at a plant and does not give trouble, it must be due to the presence of sufficient fly-wheel effect and a slow governor, which prevents any sudden load changes. The curves shown prove, however, the impracticability of the author's formulas, principally because of their disregard of the governor action.

The author states: "If a restricted orifice were inserted between the pipe line and air tank which caused a loss of head of 190 ft. with the full load flow (aV_1) entering the tank", the result would be for full load on: $Y = 18.7$ and $p_2 = 346$.

If the head were allowed to drop instantly 190 ft., the normal full-load draft velocity would have to be increased simultaneously in the proportion, $\frac{459}{459 - 190}$, in order to maintain the full load; but, if a velocity of $V = 12.2$ (full-load velocity corresponding to 459 ft. net head) requires a pressure head of 190 ft. at the restricted opening, then the actually demanded draft velocity of 20.8 would correspond to an actual pressure drop of $\left(\frac{20.8}{12.2}\right)^2 \times 190 = 552$ ft., which is more than the total available head. Consequently, a restricted orifice of

Mr. Wahlman.

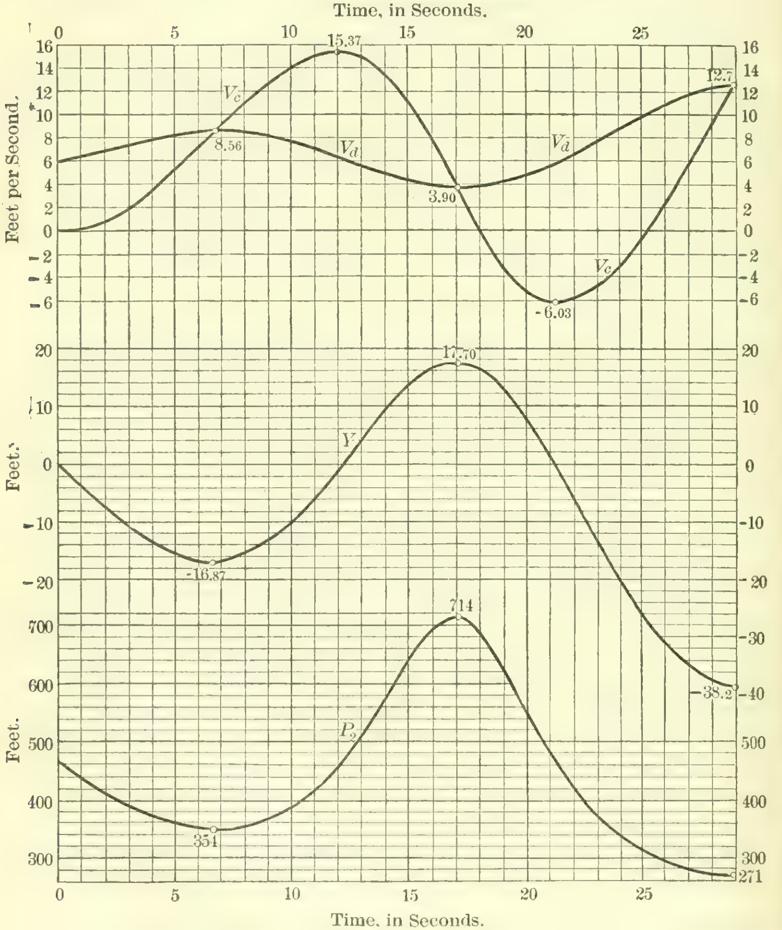

FIG. 4.

Mr. Wahlman.

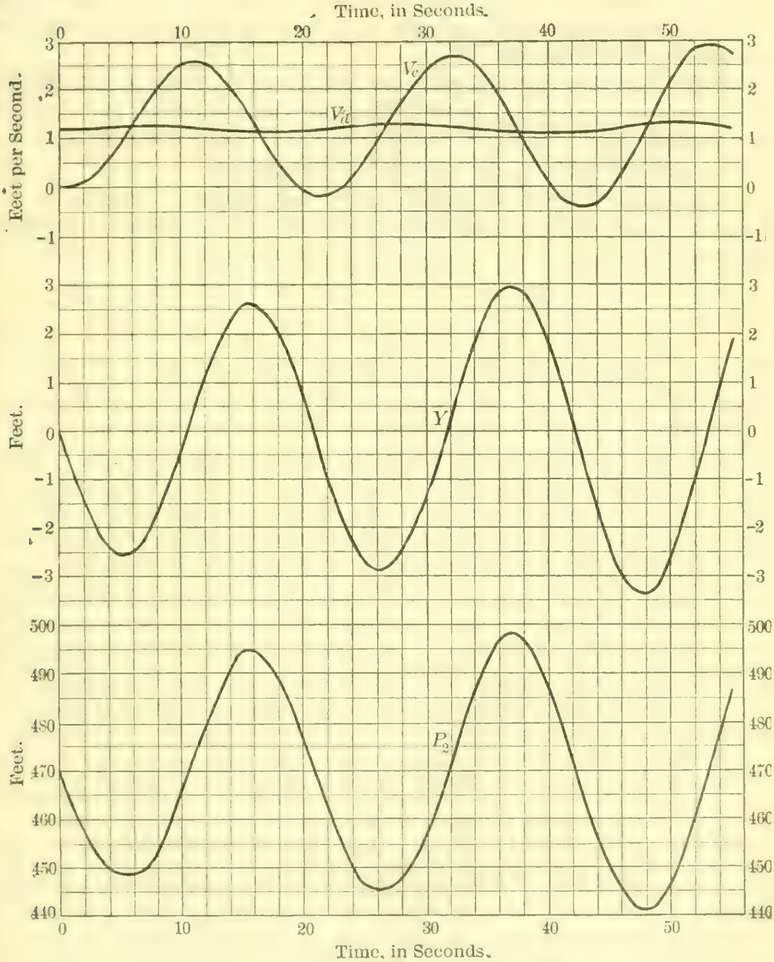

FIG. 5.

Mr. Wahlman.

the size indicated by the author would not permit full load to be thrown on instantaneously, which again shows how important it is to consider the governor action.

Should it be practical, however, to allow a $K_1 = 190$, the port opening ought to be dimensioned so as to pass a flow corresponding to a draft velocity of 20.8. Such a case, however, has not been worked out by the writer, as the velocities, as well as the drop in head, are too great to be practical.

That the restricted tank opening or differential feature is a decided improvement on the simple tank, however, the writer has had occasion to ascertain, while working out several surge-tank problems, and this applies most forcibly in the case of the air tank.

A good example of a well designed air regulator appears in the Annual Report of New York State Water Supply Commission for 1909, on page 326.

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PAPERS AND DISCUSSIONS

THE CAPE COD CANAL

Discussion.*

By Messrs. Clemens Herschel and T. Kennard Thomson.

Clemens Herschel,† Past-President, Am. Soc. C. E. (by letter).‡ —To construct and turn over for operation a public work that has been in contemplation for 300 years, assuredly makes its chief engineer deserving of recognition and praise from the profession and from the general public; but when to this is added a carefully written account of the history of that work, the details of its construction, and an engineering treatise on the underlying principles of its design and operation, the paper becomes a notable one for insertion in the *Transactions* of the Society; and deserving the thanks of its members for the work done in gathering the data on which the treatise is founded, and in writing the paper itself. Especially is all this true in the present instance, when so little is generally known, even among engineers, of tidal action; and when the paper may be expected at last to put a quietus on a delusion than which the pursuit of witchcraft was no more brainless; which is capable of doing continued enormous harm; and which is yet active among us, as will be shown. Mr. Herschel.

These preliminary remarks will sufficiently show the spirit in which the writer approaches the task of discussing this paper, and may counteract any notion of his intending it to be mere criticism. The very fact of a high value set on the paper, has engendered in the writer a wish to have it as near perfect as discussion can make it, and has caused him to contribute what he can to that end.

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* This discussion (of the paper by William Barclay Parsons, M. Am. Soc. C. E., published in August, 1917, *Proceedings,* and presented at the meeting of October 3d, 1917), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† New York City.

‡ Received by the Secretary, September 7th, 1917.

Mr. Herschel.

Perhaps the most curious historical statement in the paper is the following on page 1038:* "This contribution of General Foster's changed completely the whole character of the enterprise, because, after the publication of his report, a canal with locks was not again considered."

The writer, who was active in Cape Cod Canal matters in 1870, and for 40 years thereafter, states that the complete reverse of this is true. The charter of 1870, antedating General Foster's report, contemplated a Cape Cod Canal without locks, and no charter granted since then ever was or yet is absolutely secure from having locks foisted upon it. The trouble that prejudice and ignorance are capable of accomplishing in the affairs of men is not so easily disposed of as the paper before us would cause us to believe; and now for the proof.

The following is from the letter-press book of the writer; and will the reader please note the date:

"BOSTON, January 18, 1870.

"THOMAS LAMB, ESQ.,
"Prest. N. E. National Bank,
"Sir,

"Agreeable to your desire, I have examined,—to no great degree of completeness however,—into the subject of a Cape Cod Ship Canal. I have no doubt that a single system of locks, as you suggest, is sufficient to operate such a canal and is preferable, in more than one respect, to two systems of locks,—one at each end,—and I should think such single lock ought to be placed over near the Buzzards Bay end of the Canal.

"After due reflection and some investigation, however, I think the Committee appointed by the Senate and House in 1860, in their Report of November 5, 1862, make a sound suggestion, when they speak of the propriety of investigating,—which has never been done,—whether a canal without locks could not be made; by making the same, say, 20 miles long, instead of $7\frac{3}{4}$ miles, as then surveyed.

"This would be doing in canal-building only what has been done on every Railroad that ever was built; also on ordinary canals, etc.; that is, increasing the length, to diminish the descent or fall per unit of length; and I find that such a canal twenty miles long need not have a greater surface velocity than 2.2 miles per hour, or bottom velocity so great as 3 ft. per second, at the times of the greatest difference of level between the two Bays.

"In case of a canal confined by artificial banks and having a current of 3 ft. per second flowing through it for a considerable length of time, this velocity would give good grounds for apprehensions, and it might be impracticable to make the banks firm enough to resist it; but with a canal excavated in the natural soil, below the level of the surrounding country as this would be, and having the maximum velocity only twice in 24 hours for a short time, the difficulties of the case can, I think, be boldly met and overcome by simple precautions or remedies, to enumerate and describe which would make this com-

* *Proceedings*, Am. Soc. C. E., August, 1917.

munication too long and elaborate. To give you an idea as to what rank such a velocity of current is entitled to, it may be stated that the same and greater velocities exist in multitudes of inlets found on every coast where the tide flows in and out. High waters and freshets in rivers have velocities as great as 12 ft. per second and over. Mr. Herschel.

"I am satisfied that the cost of a longer canal without locks would be about the same as that of the short one with locks and artificial harbors; that it would cost much less to operate, and would be more used by seamen. Further, we are in the one case compelled to accept of some inferior harbors or sites for such, as termini of the canal, while in the other, we can select good harbors to be the vestibules, if I may so call them, to either end of the canal; and in this one item alone I am satisfied that millions could be saved.

"Very respectfully yours,

"CLEMENS HERSCHEL,

"Civil Engineer."

The "sound suggestion" referred to, is the one quoted on page 1037* of the paper (page 54 in the original report), beginning:

"It may not be unreasonable to suppose that some method of avoiding the force of the currents might be discovered without the cost and delay of using locks and gates."

As stated, the letter above quoted was written after only a very brief consideration of the subject, and errs on the side of safety, in the endeavor to keep at a minimum those terrible currents, which, in the popular belief of the day (to some minds yet existing), were going to work havoc with the geography of the "Old Bay State."

General Foster's report is dated May 10th, 1870. The Resolve of the Massachusetts General Court, in consequence of which General Foster's views were called for by a Congressional Committee and by the Chief of Engineers, is dated April 2d, 1870. The charter of 1870 is dated February 26th, 1870.

To the best of the writer's recollection, he brought to General Foster's attention, and advocated directly to him, the advantages of an open sea-level canal; a form of canal, which, at that time, and often since, somehow or other, has been unpopular; but which, in most or all cases of a tidal canal, is the best form, and will no doubt hereafter generally prevail.

It was General Foster's merit, however, to cut loose from the unfounded fears of a short-line Cape Cod Canal, one built on a route only 8 miles long, and, by computations, to show that the surface slope to be expected on such a canal, and their consequently engendered velocities, need not be feared as rendering ordinary steam navigation impracticable. His report also points out, in conclusion:

"* * * that it may, and should be made an open canal, without locks, in order to accommodate the greatest number of vessels that may present themselves for passage, and at the same time keep the

* *Proceedings*, Am. Soc. C. E., August, 1917.

Mr. Herschel. canal clear of ice" (and sand, he might have added). That he adds, "* * * that a breakwater at the eastern terminus is necessary as a protection to the mouth of the canal, and important as a harbor of refuge for vessels navigating the bay" (meaning a breakwater, at right angles to the centre line of the canal, out in deep water of Sandwich Bay); and: "It is also proposed to provide, as a precaution against the effects of very high tides, guard gates at each end of the canal, which may be closed in an emergency to avert danger to the Canal, but which are ordinarily to be open"; with an allowance of about 3⅓ million dollars for breakwater and guard gates; that he adds all this is only "throwing a tub to the whale" of popular clap-trap. But there was and is one engineer, at least, who never could stomach prejudice and self-assertive ignorance, come what may, and he kept up the contest for an open sea-level canal without breakwaters, or guard gates, or any other gates, as will presently appear, until, due to the enterprise of August Belmont and others, he had the great pleasure of navigating just such a canal through Cape Cod in 1914.

Before General Foster's day and since, many engineers have considered it politic, expedient, leading to profit, what not, to defer to the cry of the populace.

General Foster could hardly avoid estimating for the breakwater, because the Resolve above referred to, specifically called for a breakwater; and the demand for safeguarding the canal from those terrible currents, was very strong.

General Foster left Boston in 1872, and died in 1874, and thus was prevented from actively furthering the construction of any Cape Cod Canal; as he undoubtedly would have done, had he lived.

There were, of course, certain "interests" that did not want the canal built. The freight traffic between New York and Boston, by railroad; by the mixed railroad and steamboat lines; and by one "outside" line, was very comfortably fixed; and ready to cause others to let well enough alone. It was easy to slip a clause into the several charters, looking to the construction of locks in, or enabling certain Boards to force them upon the canal; all of which would with almost certainty cause that charter eventually to lapse. For a canal, frozen up every winter, built to facilitate winter traffic around the Cape, is palpably an absurdity; let alone other most serious disabilities of such a canal, as will presently appear.

Does any one think that these absurdities, and arguments for having them come into being, are extinct? If so let him turn to page 100 of the "First Annual Report of the Commission on Waterways and Public Lands", of Massachusetts, for 1916 (issued as this is written), and read as follows: "Requisitions * * * pending a determination of the question of a lock, tidal gates or other device for controlling the current of the canal, and of other matters." Or let him scan the much

Mr. Herschel.

abused 1917 United States River and Harbor Bill, the Section relating to the purchase and making of a free channel (as it ought to be), by the United States, of the "Waterway connecting Buzzards Bay and Cape Cod Bay, Massachusetts", "with or without a guard lock."

At variance with the statement on page 1038 of the paper, the writer's direct professional relations with the Cape Cod Ship Canal Company chartered in 1870, did not begin until 1878. But in one way or another he had such relations with men and corporations interested in the project in 1870, and thereafter. In this way was written the 1878 report (forty closely printed pages), on the Cape Cod Ship Canal, and, in 1884, the Company chartered in 1883 reprinted it with additions. It is to be found in various libraries, and is believed to have been useful as a source of information on the subject ever since.

This report is too voluminous to quote. It advocated for the first time an open sea-level canal, without locks or gates of any kind, no harbor-producing breakwaters, two parallel jetties to extend the banks of the canal into the deep water of Sandwich Bay; the very type of canal built in 1909, more than 30 years later. One feature of the report, of course now out of date, but presumably of value as showing a method applicable in other such investigations, was a determination of how much the carrying of coal around the Cape, rather than landing it south of the Cape, had actually cost for the 10 years, 1868-1877, both inclusive. This was done by looking over the books of several coal dealers each in Boston, Salem, and Lynn, and comparing these freight rates paid, with those paid by several dealers each in Providence, Fall River, and Newport; doing this for every month of the 120 months considered; adjusting the difference found in proportion to the proportional shipments of coal during the several months in the year (more per month in the summer than in the dangerous winter months); and doing all this twice, once with New York (Rondout), as the shipping port; and, again, as coming from Philadelphia. Two months, in the 10 years, this difference was $1 per ton; and for 2 months it was nothing.

The true average difference was 57 cents per ton, shipping from Philadelphia, and 48 cents shipping from New York. For purposes of the report, and attempting to foretell what that difference would average for the next 10 years, 30 cents was adopted. And attention was called to the effect of shipping in tows of barges, etc., etc. Since then, the shipments of coal around the Cape have increased about four-fold.

To justify a conviction that those currents that have been spoken of would not saw the earth in halves, beginning with the slight notch cut on its surface by dredges, and then allowing the tides to act, the author of the 1878 report also wrote an article,* more particularly

* *Journal,* Franklin Inst., May–July, 1878.

Mr. Herschel. for engineers' reading, entitled "On the Erosive and Abrading Power of Water."

It may now be thought that this discussion exaggerates the fear held by some, and studiously engendered in others, as to the effect of the expected canal currents on the Canal. To illustrate what it was, the writer remembers that a colleague and sort of running mate, half surveyor, half astronomer, of "Professor" Mitchell, referred to on page 1036, publicly denounced the project of an open canal, by saying that he would "as soon think of turning loose Niagara across the State of New York;" very effective as a piece of rhetoric, no doubt, but less valuable as an item of engineering judgment or knowledge. In February, 1901, the writer was engaged to argue, and did argue, before the Harbor Commissioners of Massachusetts in favor of allowing the canal to be built as an open sea-level canal. Other occasions for the writer's activities in favor of an open sea-level canal at Cape Cod than those mentioned came to pass during the 40 years, 1870 to 1910, but it will be needless to refer to them.

Passing now to Part II, The Hydraulics of the Cape Cod Canal, a general review of the situation must bring out, in strong relief, the usual situation with regard to the practical hydraulic problem of the flow of water in artificial channels. Formulas based on data observed in any actually built channel or channels are only approximately applicable to other such channels, because none of the channels mentioned is ever mathematically uniform or regular or exactly similar, in construction; and when based on data procured from specially constructed regular channels, is only approximately applicable (by reason of irregularities found in them), to channels as they are found in actual practice.

A classical example of the first part of the stated proposition is that furnished by a comparison of the discharge of the Sudbury Conduit, built 1873-1883, supplying Boston, and that of the last-built Croton Aqueduct, 1887-1895, supplying New York City:

"These two works were built under practically identical leadership, and under precisely similar circumstances and conditions, the one immediately after the other. The same engineers, the same class of materials, identical methods of construction, distinguish them. Most excellently conducted gaugings or experiments of discharge were made upon the Sudbury Conduit, and its formula of discharge was computed when it was new. Yet when such measurements were repeated on the newly completed Croton Conduit, only 94.5% of the expected results were attained."*

In a measure, this weakness, or inexactness of hydraulic formulas, is recognized throughout the paper; as is also the fact that, in computing for a navigation canal, an exact mathematical evaluation of

* See p. 77, "115 Experiments", by the writer. John Wiley and Sons, 1897.

Mr. Herschel.

expected currents, is of no practical value. "The average skipper cares very little whether he has to tow (or run) against 3.3, or 3.6, or 3.7, (or 4 or 5) knots per hour, for a distance of 8 miles."* The several given methods of computing probable future velocities in a tidal canal are, however, of value in giving engineers their choice of which they will use, and of catering to the most fastidious. For those who like computations to the third decimal figure, based on data that are not exact to one decimal figure, some of the formulas referred to are of the kind they like. Also of value in demonstrating that the results of computation by any and all of the formulas given, differ only within reasonable limits. If formulas for uniform flow in masonry conduits, as above mentioned, differ by 5.5%, what can be expected of formulas that deal with tidal flows in irregular excavations in sand and gravel? In the first case, million gallons per 24 hours actually discharged into reservoirs are sought after; while in the latter it is only a question whether the sides of the canal can be made current-resisting, and the canal itself navigable, between the limits of "slack water", and a maximum to be expected velocity.

A word more as to navigability in currents of different value. Sailing vessels in tidal canals can be left out of consideration. They and small boats can, if permitted, drift through with a favorable tide, at the proper hours, or hire "tows." Steamers take care of themselves, or may be assisted through. As a matter of fact, the channels of commerce of the world abound in swift water currents, as shown in a record† of such, prepared by the late President, Am. Soc. C. E., Elmer L. Corthell. They vary, materially: 7.5 knots in the St. Lawrence River; 10 knots in Portland Firth; 13 knots in the Gironde, France; and a multitude of others. On rapids in the Rhine, Elbe, and Rhone Rivers, (more than 9 knots), cable towing has been in use for many years. It was proposed, if found necessary, for the Corinth sea-level canal, but was not found necessary.

An along-shore fisherman, or navigator, or draw-tender, who has known the Monument River, man and boy, for 30 or 40 years, is, of course, very much astonished, or may even feel outraged, on viewing its ancient sea channel, at an hour, perhaps, when there is a 4-knot or even a 5-knot current to contend with, to see the tide now running there. Had he fished, navigated, or viewed the Harlem River, we will say, between the Hudson and the East Rivers, the same length of time, he would think nothing of it.

The investigations of Mr. Corthell and of his predecessors showed clearly that uniformity of cross-section, at least in a hydraulic sense, (uniform mean hydraulic radius, and uniform depth, also a good alignment), were the great elements making for good navigability.

* *Journal,* Franklin Inst., May–July, 1878.

† *Memoires*, Société des Ingénieurs Civils de France, 1906.

Mr. Herschel. Nothing so sensitive as a constriction, even only a slight one, in a tidal channel.* Any one who has worked in a tidal stream, with, say, a 10-ft. tide, and has seen a roaring tideway quiet down to slack water inside of about 2 min. by the watch, only to turn around and become a roaring tideway in the other direction inside of another 2 min., will ever forget it, and what it teaches. All changes of cross-section must be made very, very gradually, as has been spoken of in the paper under discussion.

The same considerations also make little better than arrant nonsense of the constantly recurring proposition, sometimes in very high places, to put "guard-gates", "tide-locks", what not, into an open sea-level canal, to be used only occasionally. It would be impossible or impracticable to make such guard-gates or locks, of an equal hydraulic radius and depth and area as the main canal; or, if so made, to connect it to the main channel gradually, and by sufficiently slow degrees. The consequence would be that the tidal wave would at once pile up at such a constriction, making there a point of dangerous navigation, back and forth, as just referred to. And as for "controlling" the currents in such a channel, by the operation of such gates, some of the time, that again is wholly impracticable. A "bear-trap" dam at each end of the canal might do it, but would not be navigable; and no "gates" of any kind could with safety to the gates be operated in such a canal, with or against the current.

To the writer, it appears that "the excellent Homer nods" on page 1130. Times of high water at Stations 410 and 35, surely do not control, nor are they in the main the results of, velocity of wave propagation in the canal. Times of high water at either end of the canal are almost exclusively dependent only on times of high water in the ocean bays at either end, and are very little influenced by wave propagation, positive or negative, both of which take place, through the canal. The writer has great respect for the formula given on page 1129, $w = \sqrt{g\ H}$, having himself by experiment* tested it. In his opinion, it no doubt acts in full force and continuously from both ends of the Cape Cod Canal, though, on only 8 miles of length, it has small opportunity for producing any notable or material effects.

Again, on page 1147, picking out "a stretch of uniform or nearly uniform cross-section", "to test the validity of the equations supposed to furnish the values for the elevation of the water and velocity at any section and time" will not overcome the difficulties of the case. These elevations and times, in the case of a tidal canal, oper-

* See the paper "On Waves of Translation that Emanate from a Submerged Orifice, Together with an Examination of the Feasibility of the Proposed Baie Verte Canal", *Transactions*, Am. Soc. C. E. (July, 1875), Vol. IV, p. 185, by the writer.

ated, as it is, by the great outside ocean tides, are functions of every foot of the length and depth of that canal, and of its varying cross-sections, in the open and under bridges, etc., from ocean to ocean. Mr. Herschel.

In spite of what has above been said as to methods of computing, the writer ventures to add what would have been or would be his method of computation, had he been or were he called on to compute water heights and velocities in a long sea-level canal. It calls for no formulas except $v = C \sqrt{R\ S}$ and $w = \sqrt{g\ H}$, and may be gleaned or inferred from an article of his, entitled: "On the Solution, Mainly by the Aid of Graphical Construction, of a Problem in Practical Hydraulics."* The diagram accompanying this article is herewith reproduced as Plate XLIII.

It will be noticed that the method referred to is based on assuming that the ocean tide rises, not in the form of a sine curve (when plotted with respect to times), but in steps following a sine curve in outline. Then, by taking the "tread" of these steps as short as may be desired, any desired degree of coincidence with a sine curve rise of tide may be attained.

There are now in existence and recorded in engineering literature, so far as the writer knows, just five open sea-level canals:

Suez Canal,
Corinth Canal,
East Bay Neck Ship Canal,
Pamlico Sound-Beaufort Canal,
Cape Cod Canal.

The "Kiel Canal" would be another, only that, for purposes of marsh drainage along its line, it was necessary to put in lock gates at either end, and keep the canal level at mean level of the sea.

The first three named have been discussed by the writer,† together with the, at that time, only projected Panama Canal. So was the Cape Cod Canal at that time, only a projected canal; likely to remain a mere project for another century or so, by all the unkind (as we will call them) things said of open sea-level canals in reports on the Panama Canal. A contention in favor of an open sea-level canal at Cape Cod can be read between the lines of much that the writer then wrote about a sea-level canal at Panama; and proof direct could be given, how some of the intended decrying of open sea-level canals, intended to bolster up the agitation in favor of a lock canal at Panama, was designedly brought to naught, as a protection to a future building of an open sea-level canal at Cape Cod.

That the Panama Canal was built as a lock-canal should disturb no one. The hydraulic questions involved were admittedly never

* *Journal*, Franklin Inst., 1871-72, pp. 105 and 181.

† *Transactions*, Am. Soc. C. E. (June, 1906), Vol. LVI, p. 206.

Mr. Herschel. properly considered, for the reason, as given, that "the limited time available to the Board [of Consulting Engineers] has not permitted the full consideration of this question, which is desirable;" a polite shirking of duty in an important matter, probably without parallel in engineering history.

It has now been given to Mr. Parsons to demonstrate by physical facts, and by the first comprehensive treatise on sea-level canals written in the English language, accompanying a work of which he was the Chief Engineer, that sea-level canals are amenable to common sense and to engineering computations. This, and the fact that he was one of the three American engineers on the Board of Consulting Engineers (William H. Burr, M. Am. Soc. C. E., and Gen. George W. Davis were the two others) who, in 1905, did not join in the hue and cry for a lock canal at Panama, but voted with the majority of the Board composed of these three, together with all the well-informed British, French, Dutch, and German engineers selected by their respective Governments as life-long experts on the questions involved, make up a record of which any hydraulic engineer may be proud, and to which the writer believes time will only add high repute and renown.

(Mr. Herschel attended the meeting at which Mr. Parsons' paper was presented; he did not read the foregoing written discussion, but addressed the Society as follows:)

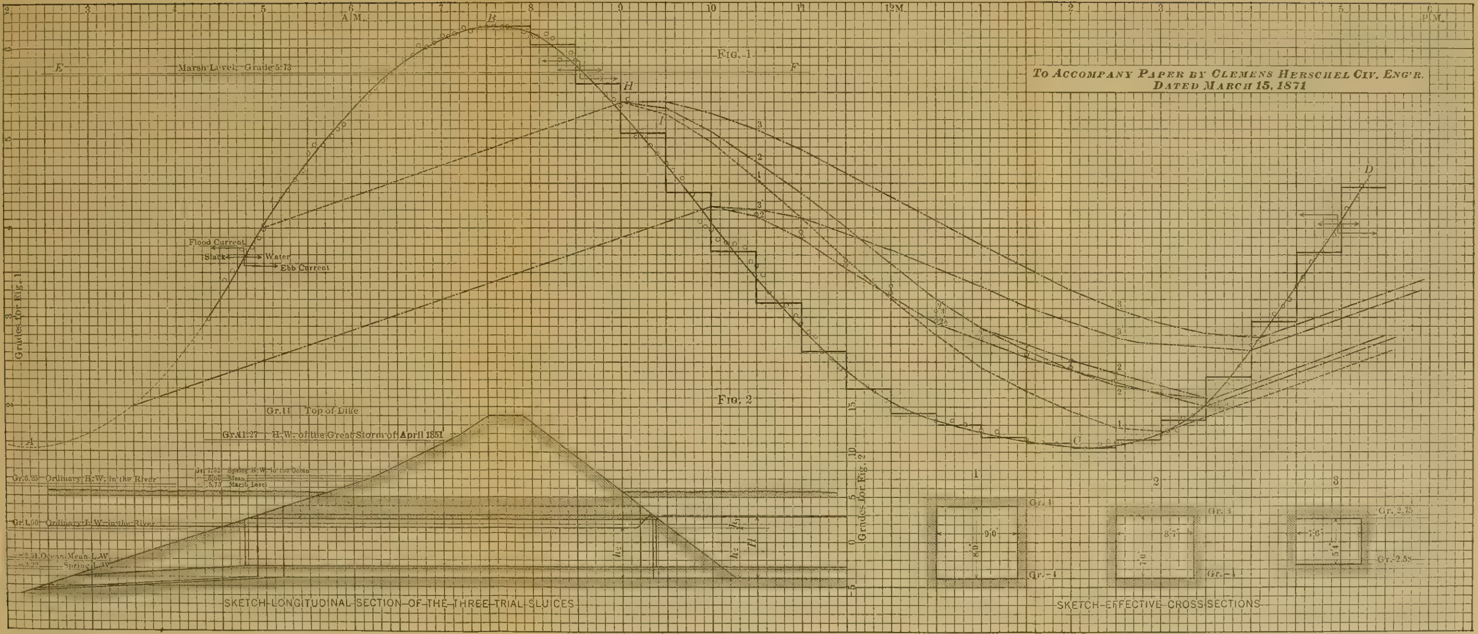
The speaker has written a discussion on this paper, but, instead of reading it, he will consider the Cape Cod Canal from another angle than that from which he viewed it in the foregoing discussion.

This paper is very important, for more reasons than one. If one looks at a map of the United States and follows carefully the inland, or nearly inland, waterways along the east coast, one will soon come to the conclusion that it would not require much of an engineering undertaking to enable a vessel to go from Massachusetts Bay to the Gulf of Mexico by an inside passage. It is almost, or wholly, possible now for very small craft, and one can see clearly that, before a hundred years have gone by—or possibly before 20 years—that route will be open for vessels of possibly 25 or 30 ft. draft.

Now, this Cape Cod Canal may be considered as the first link in that chain of communications, important for freight traffic, and of the greatest importance as a measure of national defence.

There is in existence an association, called the Atlantic Deeper Waterways Association, which has as its object the construction of an inside waterway all along the eastern coast of the United States, and for the last 10 years it has advocated this project. A Member of Congress, J. Hampton Moore, seems to have made this his life work, and every year this association has excursions over and inspections of parts of this waterway. Thus far, that seems to have been the only method

GRAPHICAL METHOD APPLIED TO A PROBLEM IN HYDRAULICS



of agitating the subject. It is hoped that in the course of time the prosecution of the public works of the United States will be carried on in a more rational way than by pulling and hauling and junketing and picnicking. All are agitating for such improved and more rational ways of selecting needed public works, and there is good reason for believing that we are on the very threshold of a rational method of making such a selection. Mr. Herschel.

It may not have been noticed by all engineers, but the last river and harbor bill had incorporated with it a clause providing for the formation of what has been called the Waterways Commission. The speaker thinks that this clause has been so little noticed that he will venture to read what this waterways commission proposes to create and do. It has been enacted:

"That a commission, to be known as the Waterways Commission, consisting of seven members to be appointed by the President of the United States, at least one of whom shall be chosen from the active or retired list of the Engineer Corps of the army, at least one of whom shall be an expert hydraulic engineer from civil life, is hereby created and authorized, under such rules and regulations as it may adopt, to bring into co-ordination and co-operation the engineering, scientific and constructive services, bureaus, boards and commissions of the several governmental departments of the United States and commissions created by Congress that relate to study, development or control of waterways and water resources and subjects related thereto, or to the development and regulation of interstate and foreign commerce, with a view to uniting such services in investigating, with respect to all watersheds in the United States, questions relating to the development, improvement, regulation and control of navigation as a part of interstate and foreign commerce, including therein the related questions of irrigation, drainage, forestry, arid and swamp land reclamation, clarification of streams, regulation of flow, control of floods, utilization of water power, prevention of soil erosion and waste, storage and conservation of water for agricultural, industrial, municipal and domestic uses, co-operation of railways and waterways, and promotion of terminal and transfer facilities, to secure the necessary data, and to formulate and report to Congress as early as practicable a comprehensive plan or plans for the development of waterways and the water resources of the United States for the purposes of navigation and for every useful purpose, and recommendations for the modification or discontinuance of any project herein or heretofore adopted."

Then follows a provision for pay and compensation of members, and other clauses which it is not necessary to read; but that gives one an idea, and the speaker thinks that all will agree with him that the intention of the draftsman was at least comprehensive, and that we may hope that some good will come of it.

Now, the Cape Cod Canal is a public work, completed by private enterprise, which is directly in line with this clause which has just been

Mr. Herschel. quoted, and it is to be hoped that this commission will continue the good work.

A right of way for a sea-level canal, without locks, from New York Harbor to the Delaware River has been promised by the State of New Jersey; the routes have been surveyed by the United States Engineers and no difficulties have been found. There is a canal now from the Delaware River to Chesapeake Bay, and so on. There are several other small canals that could be widened and deepened. One of these, on the same line, is already a reconstructed sea-level canal; and, for such reasons as the speaker has given, he thinks this paper of great importance, and that the uses of the canal will grow as time goes along.

Mr. Thomson. T. Kennard Thomson,* M. Am. Soc. C. E.—The Society is much indebted to Mr. Parsons and Mr. Douglas for a well-prepared and well-presented paper on this interesting work, and Mr. Belmont and his associates deserve the thanks of the entire country for carrying through a much needed public work as a private enterprise.

It can easily be understood why it was necessary to economize on width, depth, etc., but, now that the enormous future value of this canal must be apparent, the Government should give every aid and encouragement to the efforts to obtain an ultimate depth of at least 40 or 45 ft., an ultimate width of at least 1 000 ft., and, eventually, instead of bridges, tunnels carrying traffic under the canal. These figures may sound fantastic, but they are not; although the speaker may be laughed at now for seeing "too much", he may also be laughed at by future generations for not seeing far enough.

The Atlantic Deeper Waterways system which Mr. Herschel has explained so well (the speaker has also been a member of the Deeper Waterways Association for some years) will eventually result in deep inland waterways from Boston to Florida. It is true that, at present, the Hon. J. Hampton Moore only contemplates a depth of 12 ft., or less, in some places, but, as soon as these depths are obtained, greater and greater depths will be insisted on. A glance at a map of the Atlantic Coast will astonish most people by showing how comparatively few links of canals will be required to complete the inland waterways from Boston to Florida and from New York to the Great Lakes.

The speaker takes this opportunity to "report progress" on his project for "A Really Greater New York", which will afford the most important link in this chain of inland waterways. After working nearly every day for six years and writing repeatedly to three Governors of New York State and two of New Jersey, as well as to two Mayors of New York City, and many prominent men, and addressing this Society on more than one occasion, the speaker is delighted to state that real progress has been made, for Governor Whitman and

* New York City.

Governor Edge have appointed a joint board to make this harbor proposition an interstate affair. These men are: Messrs. William R. Willcox, Chairman, J. Spencer Smith, Vice-Chairman, E. H. Outerbridge, Arthur Curtiss James, De Witt Buskirk, and Frank R. Ford, with Gen. Goethals for Chief Consulting Engineer. Mr. Thomson.

The speaker has photographs of a suction-dredge, having a discharge pipe 42 in. in diameter, now used in Egypt. It was designed by A. W. Robinson, M. Am. Soc. C. E., undoubtedly the greatest authority on dredges. The actual work of this dredge has been 50 000 cu. yd. per day, at a cost of less than 1 cent per yd.

When the "Really Greater New York" project is started, we will probably have several of these machines and will be glad to lend them to the Cape Cod Canal—between times—to widen and deepen the canal as will be required.

The speaker does not think it safe to assume that there will be no danger from the teredo at Cape Cod, for he was called in a few years ago to examine a bridge at Fall River, where one pier had settled 2 ft. over night. This condition was due to the teredo destroying the piles within two years of the time the bridge was built.

Many think that the teredo attacks only at or near the surface of the water. In this case, however, the piles had been cut off 45 ft. below the surface, and the pier (granite face and concrete backing) had been built on a 4-ft. grillage, as the grillage was sunk to the tops of the piles. The speaker saw one of the pile heads which was brought up and split open, showing both live teredo and live limnoria hard at work.

The pier had settled 2 ft. at one end, so a coffer-dam was built around it, concrete was forced under the grillage, and the bridge seat was restored to its proper level, with satisfactory results.

New York Harbor is now safe from the ravages of the teredo, but when the speaker's project for "A Really Greater New York", which involves great trunk sewers down the present bed of the East River, also through New Jersey to a point some 20 miles away from Sandy Hook, is accomplished, the rivers and bays of New York City will be safe to fish and swim in, and the piles of the wharves will have to be protected against the teredo.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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PAPERS AND DISCUSSIONS

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HYDRAULIC PHENOMENA AND THE EFFECT OF SPREADING OF FLOOD WATER IN THE SAN BERNARDINO BASIN, SOUTHERN CALIFORNIA

Discussion.*

By Charles H. Lee, Assoc. M. Am. Soc. C. E.†

Charles H. Lee,‡ Assoc. M. Am. Soc. C. E. (by letter).§—The author has presented a remarkably clear and concise analysis of hydraulic conditions in the San Bernardino Basin, and has added a valuable contribution to scientific ground-water literature.|| His long-term comparisons of residual mass-curves of rainfall and run-off and well and Artesian stream-flow fluctuations, are especially illuminating. The writer was much interested to find that the author, with additional data for 4 years and well records not previously available, has reached practically the same conclusions as he himself reached in 1912 after an extended study of the situation. Mr. Lee.

The conclusions which the writer draws from the paper with regard to the local conditions in the San Bernardino Basin are:

1.—That ground-water levels and the associated phenomena of Artesian pressure, Artesian well flow, and Artesian spring and stream flow, in the San Bernardino Basin, have wide natural fluctuations corresponding to the broad variations in annual rainfall and run-off.

* This discussion (of the paper by A. L. Sonderegger, M. Am. Soc. C. E., published in September, 1917, *Proceedings*, and presented at the meeting of October 17th, 1917), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† This discussion was presented before the Southern California Association of Members of the American Society of Civil Engineers, at its meeting of April 11th, 1917.

‡ Los Angeles, Cal.

§ Received by the Secretary, May 31st, 1917.

|| Report of the Conservation Commission of California, 1912, pp. 339 to 399.

Mr. Lee. 2.—That, thus far in its history, artificial extractions and absorption have had but a minor effect on these natural fluctuations, the predominating influence being natural rainfall and run-off variations.

3.—That, in the main, the supply of the Basin is not being overdrawn or depleted by artificial means, except in the northwest or Lytle Creek arm.

4.—That artificial additions to the ground-water supply of the Basin by water-spreading on the upper portion of the Santa Ana cone have benefited that supply by bringing about a quicker return to normal conditions after the drought ending in 1904, and have provided an element of insurance against a new series of dry years.

These conclusions, with regard to local conditions, the writer believes, are all in agreement with those of the author.

A point on which the author has not dwelt, but which the writer regards as an important one from the standpoint of the future history of the Basin, is the extent of the surplus which remains undeveloped. The writer believes that, as long as swamp areas persist within the boundaries of the Artesian basin, there is a large undeveloped surplus, represented by the evaporation and transpiration losses from the moist soil and vegetation. The area of swamp land, although decreased in extent about 40% in 1904, still embraced in that year more than 10 sq. miles, and has been steadily increasing since then. The annual volume of water lost by evaporation and transpiration from the 10 sq. miles of swamp in 1904, represented at least 25 000 acre-ft., or nearly 70 sec-ft., during the irrigation season. This loss is capable of development by any means that will lower the plane of saturation below the ground surface beyond the capillary limit of from 6 to 8 ft. The latter may be accomplished by complete relief of Artesian pressure, or by local drainage, either with tile or shallow pumping on an extensive scale.

Passing from matters of strictly local inteiest, the writer recognizes two general conclusions to be drawn from the paper:

1.—The residual mass-curves of rainfall and run-off are very useful in determining the natural tendencies of the water-plane and related Artesian phenomena in an underground basin. The writer would not call them safe criteria, however, without certain qualifications. First, the rainfall and run-off records should be of sufficient length to have established stable average seasonal values, otherwise the curves may be distorted too badly to interpret their meaning. Records should be at least 30 years in length, although longer periods are to be preferred. Comparison of Figs. 1 to 3 shows that slight modifications of the curve result from extending the period from 30 to 40 and 45 years. Second, the curves must be properly interpreted. It should be recognized that the zero line does not necessarily represent average conditions of ground-water level. The accumulated deficiency or excess

of rainfall or run-off at the beginning of a period under investigation is usually unknown. The author has assumed it to be zero. As a matter of fact, it probably differs from zero, and the curve as a whole should be either raised or lowered with respect to the zero line. This fact does not affect the shape of the curve, however, and it is this from which the most useful information can be gained. Third, it should also be remembered that the quantity of rainfall or run-off bears no direct relation to the volume of absorption. The depleted basin might be almost fully replenished by one very heavy year, such as 1883-84, and the absorption during succeeding wet years might be small, due to the inability of the basin to receive further accretions; on the other hand, rainfall and run-off, although large during a given year, might not be excessive with respect to the volume of depletion, and a number of such years in succession might be required to fill the basin, as, for example, the period from 1904 to date. These conditions would result in a disproportionately high peak in the mass-curve in 1892-93 as compared with 1915-16, and equal peaks would occur in the ground-water and Artesian phenomena. Examination of Figs. 7 to 14 indicates that the actual state of affairs was as just described. It is obvious, therefore, that no horizontal line can be drawn on a rainfall or run-off residual mass-curve which will accurately represent average conditions with respect to ground-water levels. The critical ground-water stages, the rapidity of change from one stage to another, and the general tendency during any period, however, are shown very clearly and accurately by such curves. Mr. Lee.

2.—Evidences of over-draft on ground-water supply exist in a basin, if during a period of several years of accumulating excess rainfall and run-off, such as that since 1904, there is not a corresponding recovery of any or all of the following: Ground-water level in wells outside the pressure area; Artesian pressure within the pressure area; Artesian flow from wells; and flow of springs or streams fed from ground-water, or the area of permanent swamp land or cienaga. The last of these phenomena the author has not discussed comprehensively in his paper. The writer regards it as a natural barometer of Artesian conditions, and one about which fairly accurate information can usually be obtained from residents in the locality. Evaporation from such an area is one of the natural outlets of an Artesian basin, and its area from time to time indicates the relative volumes of waste from the basin.

Taking up the paper more in detail, there are several minor points on which the writer wishes to comment.

The author states that "the hydraulic head at any point [in the surface of the Artesian basin] is equivalent to the difference of the topographic elevation between that point and the Artesian rim." The writer would qualify this by deducting pressure losses within the Artesian basin. There is continued leakage from the basin, and move-

Mr. Lee. ment of ground-water, with accompanying hydraulic friction losses. The line of hydraulic pressure has a slope within the Artesian basin just as in a pipe line. This is clearly shown by Fig. 6.

The author speaks of evaporation from swamps in Southern California as being 96 in., or 33% greater than from a free water surface. The writer has made extensive investigation of this subject, and believes that the author's figure is a little high. For wet swampy ground with vegetation and without an excessive accumulation of alkali salts, the writer found the losses to be about 15% greater than that from the surface of a large body of water. For wet bare soil, with the plane of saturation practically at the surface, he has found it slightly less, the quantity being more than 90% of that from a free water surface. For greater depths to the plane of saturation, the losses decrease until the limit of capillary action is reached at from 6 to 9 ft. below the surface.

The writer does not fully agree with the author in his method of determining the effect of water-spreading on the Santa Ana cone. His reasons are as follows:

1.—Comparisons of the volume of water equivalent to the annual rise or lag, in feet, which the author states is due to water-spreading, do not agree with the volumes of water actually spread.

The author ascribes an additional rise of the water plane, due to water-spreading, of 1.3 ft. over about 15 000 acres of the Santa Ana cone during the season of 1912-13. The actual volume spread was 3 066.7 acre-ft. (Table 3), which, assuming a porosity of one-third, would represent a depth of only 0.6 ft. instead of 1.3 ft., as the author determines from the fluctuations of the Williams' well. Again, the author concludes that the permanent result of water-spreading has been to raise the average ground-water level from 3 to 4 ft. over the Santa Ana cone, above what it would have been if no spreading had occurred. The average annual volume spread for the past five seasons is 12 460 acre-ft. (Table 3), of which 50% (Table 3) would have been absorbed from the natural stream if not spread. Approximately, one-third of the latter would have been absorbed below the present spreading ground near the Artesian rim, however, and would have soon escaped (Table 2). The net annual volume absorbed as a result of water-spreading, therefore, is approximately 8 000 acre-ft. Assuming a porosity of one-third, this represents a ground-water rise of 1.6 ft. over 15 000 acres, instead of from 3 to 4 ft., as estimated by the author.

2.—Conclusions based on the Williams' well alone do not necessarily apply to the whole cone. To be sure, the Williams' well is near the ground-water outlet, and its fluctuations, if unaffected by artificial draft, should be less than from places higher up on the cone. On the other hand, it is near the area in which the Gage Canal wells are situated, and is directly below and in the line of advance of water absorbed on the spreading grounds. Fully half the area of the cone is

south of the spreading grounds, and not in the direction of the steepest ground-water slope. Considering the cone as a whole, the writer does not believe that the fluctuation in the Williams' well is even an approximate measure of the average fluctuation over the whole cone. The only certain method of determining the latter is from detailed ground-water contour maps based on records at numerous wells, both within and surrounding the spreading ground, and showing the ground-water elevation throughout the cone at successive dates. Such maps would afford a simple and comprehensive basis for the solution of the problem both of the quantitative benefit of spreading and also the probable area receiving the greatest benefit, if any. Mr. Lee.

The writer, however, does not wish to be understood as taking the position that the Williams' well does not show the effect of water-spreading. On the contrary, he believes that this record affords evidence of the type he failed to find in 1912, namely, observations of ground-water fluctuations in the vicinity of the spreading grounds. The point on which he differs from the author is in the quantitative effect which water-spreading has had on the general rise of the plane of saturation on the Santa Ana cone. The writer would be inclined to place the actual annual average ground-water rise due to water-spreading at from 1 to 2 ft., instead of from 3 to 4 ft., as does the author. This rise is greatest within the spreading grounds and directly west thereof, being comparatively small to the north and south of the spreading ground.

The important point which the author has brought out, and with which the writer agrees, is that, as a result of water-spreading, an average annual volume of 12 000 acre-ft. has been absorbed on the upper Santa Ana cone, of which 50% would otherwise have flowed immediately to the ocean as flood water, and approximately 17% would have been absorbed so low on the cone that it would have rapidly escaped into the stream. The net result is the annual storage of at least 8 000 acre-ft. of water at a point on the cone from which it cannot escape for a period of 2 years or more, and is thus available as a dry-year reserve for pumping or Artesian draft from wells. The writer heartily agrees with the author that water-spreading should be confined to the upper cone, and should be carried on in dry as well as in wet years. The greater the volume of flood and winter water which can be thus stored in the gravels, the more valuable will this work become.

The writer notes with interest the author's data with respect to the results obtained by the three different methods of water-spreading. The relative simplicity and low cost of surface spreading would seem to indicate that the use of shafts is not desirable unless the area of the spreading ground is restricted. The quantity of water which the author states was absorbed by each shaft is less than that which the writer understands was absorbed by similar shafts in Lytle Creek

Mr. Lee. Canyon. The writer was informed in 1912 by Mr. C. M. Racor, Engineer for the Fontana Development Company, that each of these shafts absorbed from 1 to 2 sec-ft.

The writer has had opportunity to investigate a number of the ground-water basins of Southern California during recent years, and has found that not all have recovered from the low-water conditions of 1904. The great coastal plain Artesian basin, for instance, remains generally in the same condition as in 1904, and, locally, the height of the water plane and Artesian pressures have diminished to even a greater extent than in that year. This is due in part to the extensive pumping developments since 1904, but more particularly to the wasteful practice of allowing Artesian wells to flow without restrictions during the winter. These wells are situated principally along the lower edge of the basin, and the water thus escaping serves no useful economic purpose.

Another basin which has not recovered and, in fact, has steadily fallen since 1904, is the Perris Valley. This is apparently due to overdraft by pumping. Much of the water thus developed is used outside the valley.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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PAPERS AND DISCUSSIONS

DETENTION RESERVOIRS WITH SPILLWAY OUTLETS AS AN AGENCY IN FLOOD CONTROL

Discussion.*

By Messrs. Arthur E. Morgan, Alex. Rice McKim, and T. Kennard Thomson.

Arthur E. Morgan,† M. Am. Soc. C. E. (by letter).‡—At the time of his last visit to Dayton, about a year ago, Gen. Chittenden realized that he had not much longer to live, and stated that he considered his services to The Miami Conservancy District to be his last engineering work. He made a deep impression on the engineers and officers of the District, most of all because of his fine personal qualities. Mr. Morgan.

Gen. Chittenden was first called to the service of The Miami Conservancy District when the feasibility of the retarding basin system was under discussion. He was persistently quoted at that time as being opposed to such a system, and his published writings indicated a disbelief in the value of permanent reservoirs, at least, as a generally acceptable means for flood control. Therefore, though he was personally unknown to the engineers and officials of the District, it seemed probable that he would supply a definitely critical attitude which would tend to develop any essential weakness in the proposed plan.

His first visit was coincident with that of several other engineers. Later, he returned alone, and spent more than a month in a detailed examination of the design. As he was unable to walk, his work was

* This discussion (of the paper by the late H. M. Chittenden, M. Am. Soc. C. E., published in September, 1917, *Proceedings*, and presented at the meeting of October 17th, 1917), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† Dayton, Ohio.

‡ Received by the Secretary, October 17th, 1917.

Mr. Morgan. done almost entirely in his room, though he visited by automobile the sites of practically all the works. Mrs. Chittenden accompanied him on all his trips to care for his physical needs.

His mind was exceedingly active and restless, and he worked steadily from 8 to 12 hours a day for 7 days in the week. His demand for data from the Engineering Department seemed never to be satisfied. During the time he was with the District, the programme of the Engineering Office was interrupted to a considerable extent by his omnivorous appetite for more information. From 6 to 12 men were commonly employed in arranging and classifying data and in making calculations for him, and a considerable part of each day was spent in interviews with men in charge of the various investigations. When he had finally satisfied himself as to its soundness, he became an active partisan for the plan.

To the writer, the object of this paper seems to be, not so much to add to existing engineering knowledge, as to attempt to keep open a field of opportunity. The development of The Miami Conservancy District has established the fact that, in this instance, flood control can be secured best with retarding basins, but that their dependable use for flood control excludes their use for other purposes. The habit of following precedent is so strong that the situation in this case is apt to be used as proof that the same constructions cannot be used in any case for more than one purpose. Apparently, the author was inspired by a desire to prevent such a premature conclusion, and to keep open the field of inquiry in other cases where similar limitations do not exist. A knowledge of the dogmatic attitudes which have been taken by engineers on the subject of flood prevention would seem to justify this effort.

1.—Combined Purpose Dams in Miami Valley.—A few points in the paper seem to merit special mention. First, it may be well to repeat that The Miami Conservancy District made no mistake in not trying to combine flood prevention with power development. Of the five dams to be built by the District, the Englewood Dam on Stillwater River offers by far the best opportunity for adding to the flood-regulating capacity of the basin storage capacity for power development. If the combination of purposes is not feasible at Englewood, it is not feasible anywhere within the District. Estimates were made of the income which would be derived if the lower third of the Englewood Basin were used for permanent storage to supply a power plant, while the upper two-thirds was used for flood control. The part of the total cost of the dam which must be charged to power development in such case is more than $500 000; that is, such a dam would cost $500 000 more than one with equal flood storage, but with no power storage provided. The drainage area of the Stillwater River is 650 sq. miles; the low-water flow is about 25 sec-ft.; the regulated flow with such a dam

would be about 300 sec-ft.; the head would vary from 40 to 70 ft.; and, with a liberal over-all efficiency of 80%, about 1 500 continuous horsepower could be developed. In this locality, served by large steam central stations, the power would be worth not to exceed 9 mills per kw-hr. At this rate, the gross annual income would be $88 000. To sell the power at a load factor of 40% a plant installation of 3 750 h.p. would be required, which would cost not less than $350 000. The yearly cost of the power would include such items as the following: Mr. Morgan.

Interest 5% on $850 000....................	$42 500
Depreciation, 5% on $350 000...............	17 500
Taxes, 2% on $850 000......................	17 000
Operating expenses and maintenance........	5 000
Annual cost of power....................	$82 000

The storage of water for power would permanently submerge 4 200 acres of the best farm land in Ohio, of which at least 3 500 acres will retain practically its full agricultural value under the present plan of the District. The annual rent of this land is from $6 to $10 per acre, making the total annual cost of the power range from $103 000 to $117 000, against the liberal gross income of $88 000.

It is doubtful whether it would be a sound business undertaking to develop water power in this locality if the total annual cost of the power produced would exceed 5 mills per kw-hr. of delivered energy. At the Englewood Dam, according to the foregoing deductions, its cost would be from 10½ to 12 mills. The conditions at the other four dams of The Miami Conservancy District are still more unfavorable for power development.

2.—Extent to Which Retarding Basins are Feasible.—Gen. Chittenden's paper might seem to infer that cases are rare in which retarding basins may be used profitably for flood control. Although the majority of flood-control problems must be solved by other methods, yet the aggregate number of cases in which retarding basin control is feasible is greater than first impressions would indicate. Over the country there are probably hundreds of cases where this method will finally be found most advantageous. The following are two typical examples that have come within the writer's experience.

The St. Francis River, in Missouri, drains a mountain water-shed of 1 500 sq. miles, and will have a maximum flood run-off of not less than 150 000 sec-ft. Through the lowlands along the Arkansas and Missouri State line the river channel has a normal capacity of less than 2 000 sec-ft. By means of an extensive system of levees, this capacity is being increased to 10 000 sec-ft., or more; but, after being improved by an extensive levee system, the channel will still have a capacity of only about one-tenth of the maximum discharge from the

Mr. Morgan. hills. The control of maximum floods by levees is entirely impracticable. Just above the point where the river leaves the hills is a possible dam site, where a complete control of the river can be secured at an expense which would not be a serious burden to the land affected.

The Coldwater River, in Mississippi, drains 1 000 sq. miles of hill land and then flows for many miles through the rich alluvial lands of the Yazoo Delta, the largest and most fertile body of cotton-growing land in the United States. This river, where it leaves the hills, will have a maximum flood flow probably in excess of 125 000 sec-ft., but the channel at that point has a capacity of about 900 sec-ft. Flood control of the Coldwater by channel excavation is entirely impracticable, and control by levees is greatly complicated by numerous branch streams which enter it along its course. There are two reservoir sites in the hills, the development of which would completely control maximum floods on the river. In this case, a dual purpose dam might be built with the object of storing water for rice irrigation. In these and other instances which have come to the writer's attention, where the disparity between channel capacity and flood flow is extreme, retarding basin control may prove to be the only feasible method.

3.—Spillway Versus Conduits.—The object of the paper, as stated before, seems to be to keep open the field for the combined purpose reservoir, and not to settle any of its details. Yet a casual reading might lead to the impression that in such reservoirs the flood openings should be spillways and not conduits. The function of the spillway as a safety factor for almost every dam should be fully recognized, but the openings for flood regulation wherever possible should be conduits and not spillways. The degree of protection below the dam is limited by the maximum flow at any time during the flood, and this maximum flow is determined by two conditions, the capacity of the openings and the capacity of the basin above the dam. The ideal control would be secured with openings which would provide a uniform rate of flow throughout the flood in the river channel below the dam, and this uniform flow should be of such volume that it would pass all the water which could not be held in the basin, below the elevation of the spillway, during the maximum possible flood. If the openings are too large, more water will be allowed to pass than is necessary, overtaxing the channel below the dam, and the reservoir will not be filled. On the other hand, if the conduits are too small to care for the maximum possible flood, the basin will be more than filled, and there will be added to the flow through the conduits a short flood due to flow over the spillway crest. The use of a spillway instead of conduits for flood regulation would have the disadvantages both of too large and too small openings. At the beginning of a flood the small flow over the spillway would be at much less than the average rate. The channel below the dam would not be filled to its capacity, and the capacity of the basin

would be consumed by unnecessary storage. At the crest of the flood, the deeper flow over the spillway with its greatly increased cross-section and head would be at much more than the average rate, and the flow below the dam would be correspondingly large. So far as variation in head is concerned, the conduit and the spillway are affected alike, but, as to variation in cross-section and total discharge, the advantage is all in favor of the conduit, and this advantage is, in fact, so great as to make a spillway uneconomical for the purpose, if it can be avoided. Mr. Morgan.

4.—Terminology.—The choice of the term, "retarding basin", for the works of The Miami Conservancy District was made after consideration of all the terms in use. The term, "reservoir", generally refers definitely to a place where water or other substance is held for future use. It was especially desired to avoid this inference, and so the word, "basin", was adopted instead. As between the words, retarding and detention, the latter commonly implies permanent restraint, and the former expresses exactly the function of the works of the District, to retard but not to stop the flow of flood waters. In planning a type of construction for which no designation has been generally adopted, it seemed better to choose a name which accurately describes the work than to use one which, to a greater or less degree, misstates the functions of the construction.

Without any doubt, in the numberless combinations of conditions met in flood prevention work, numerous cases will arise where dual purpose or many purpose reservoirs will be feasible and very desirable. It is well to draw public attention to this fact, and not to allow a field to be closed through prejudice or precedent.

Alex. Rice McKim,* M. Am. Soc. C. E. (by letter).†—The writer believes that, in some cases, flood reduction and power storage can be adjusted by building a high dam and providing for three superimposed reservoir spaces; the lowest space to form a permanent lake for the preservation of fish life; the middle space to be used for power development, controlled by gates; and the uppermost space to be utilized for flood reduction, and controlled by a waste weir. Mr. McKim.

The waste weir must be properly proportioned. In his work, the writer could find no method in use for obtaining the dimensions of a waste weir for a reservoir. So he devised the following simple formula:

$$L = \frac{2\ A\ Q\ D - H\ P}{D\ F},$$

in which, L is the necessary length of the waste weir, in feet; A, the drainage area, in square miles; Q, the average high daily run-off, in second-feet per square mile, obtained by dividing 120 by the seventh

* Albany, N. Y.

† Received by the Secretary, October 17th, 1917.

Mr. McKim. root of A; and D, the duration in days of Q, equal to $1 + \frac{1}{70}$ of the longest flow, in miles, on the drainage area. H is the assumed height of the waste, in feet; P, the acreage of the reservoir surface at the waste crest level; and F, the maximum waste flow, per linear foot, in second-feet, for H.

Mr. Thomson. T. KENNARD THOMSON,* M. AM. SOC. C. E.—The discussions on flood control by Messrs. Morgan and McKim make one glad that the redeeming feature of this Republic is, that when an experiment proves a certain procedure to be a failure, it is discarded, and an attempt is made at something better. The younger members of the Society will probably live to laugh at the crude efforts at flood control now being tried or proposed. Radically different methods will be used, and there is no reason to suppose that even the great Mississippi River cannot be controlled effectively.

These discussions also give the speaker an opportunity to report progress on his project, "Niagara Falls Junior", for building a dam in the lower rapids of the Niagara River.

Many engineers have asked how the water, which amounts to 220 000 cu. ft. per sec., can be controlled while the dam is being built. For obvious reasons, it was advisable at first not to disclose the location of the dam, but now the speaker takes pleasure in stating that it will be built on Foster's Flats, the only place in the river where there is a low shelf between the water's edge and the high bank. By using this shelf, from one-half to three-quarters of the dam can be built on dry land and carried below the bed of the river.

After this portion is built, it will be easy to divert the water from the present channel through openings in the new dam. It will then be an easy matter to complete the dam.

As the speaker was confronted with the doubts of others concerning the possibility of using the 2 000 000 h.p. of the proposed development, he wrote to the Director of the Census, Department of Commerce, Mr. Samuel W. Rogers, and asked him how much power was now used in New York State, as well as the probable rate of increase. Mr. Rogers very kindly made a most comprehensive reply to the effect that more than 3 000 000 h.p. are now used, and that the annual rate of increase is more than 300 000 h.p. From this statement, it will be seen that the normal increase for 3 years will absorb all the power which the dam can furnish on this side of the boundary line.

In addition, it may be stated that in Canada power is now transmitted successfully for 250 miles. In California, it is now transmitted 543 miles. If a circle is drawn with a radius of 500 miles, using Niagara Falls as the center, it will be found that the enclosed area

* New York City.

will include the whole or part of twenty States and two Provinces, with 60% of the population of Canada. All this population and area would be within reach of the new dam, if enough power could be generated there. Yet, as Director Rogers' letter would indicate, there will not be enough to supply the State of New York. Mr. Thomson.

Incidentally, it might be remarked that the 2 000 000 h.p. mentioned will save about 20 000 000 tons of coal per year. Surely it is time that such an enormous waste of power was stopped.

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PAPERS AND DISCUSSIONS

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THE SUBSIDENCE OF MUCK AND PEAT SOILS IN SOUTHERN LOUISIANA AND FLORIDA

Discussion.*

By Messrs. Arthur E. Morgan and Orrin Randolph.

Arthur E. Morgan,† M. Am. Soc. C. E. (by letter).‡—As there are several million acres of muck and peat lands in the United States, any information relative to their character is of material importance. Mr. Okey's paper is one of comparatively few contributions of definite value on this subject which have been made in the United States. It indicates conclusively that any project for the reclamation of muck or peat lands which ignores the probability of very great soil settlement has omitted a vital factor in the problem. Mr. Morgan.

Although Mr. Okey's studies have established the fact of very great subsidence in drained muck soils, yet few, if any, of his measurements were made in those soils where the greatest subsidence would take place. Most of his measurements in Southern Louisiana were of necessity on soils which, for a long time, have been subject to occasional overflow by the Mississippi and have received more or less silt deposit. The muck a mile from Lake Okeechobee and that near Davie, in Florida, is already partly decomposed, and, therefore, a certain amount of compacting had taken place long before the first records of ground surface elevations were made. The great interior of the Everglades is composed of a coarse brown fibrous peat, in which even a materially greater subsidence may be expected than in most of the cases recorded by Mr. Okey. Tests by burning show the ash content of Louisiana

* This discussion (of the paper by Charles W. Okey, Assoc. M. Am. Soc. C. E., published in September, 1917, *Proceedings*, and presented at the meeting of November 7th, 1917), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† Dayton, Ohio.

‡ Received by the Secretary, October 15th, 1917.

Mr. Morgan. muck south of New Orleans to be from 20 to 50% of the dry weight of the raw muck. The ash content of the interior of the Everglades, as recorded by the United States Soil Survey, is from 3 to 6%, according to the writer's remembrance.

Engineers of the Morgan Engineering Company recently spent about 6 months in making a detailed examination of more than 150 tracts of muck lands in various parts of the United States, principally in the Atlantic Coast States. It appears that the terms "peat" and "muck" are used to cover a wide variety of soils and soil conditions. In New York, Pennsylvania, and the adjoining States, there are considerable areas of muck lands formerly covered with dense growths of hardwood and cedar. This soil is well decomposed. When first drained, it is brown, but, in a year or two, it turns black and has the granular appearance of old-fashioned gunpowder. When cleared and put in cultivation this is the finest truck garden soil to be found in the United States, and such muck areas commonly sell for from two to five times as much per acre as the adjoining loam soils. The high value is due not primarily to great fertility, but to the exceptional ease with which such land may be tilled, to its unusual capacity for retaining capillary water, and to its loose texture, which makes perfect root development possible.

At the other extreme of soils of this class is the coarse brown fibrous peat, great areas of which occur in Minnesota, Wisconsin, and Florida, as well as in Canada and many parts of Europe. Nowhere does one find records of the profitable use of this soil for cultivation, except in rare cases after long and painstaking effort for its reduction. Most of this worthless brown peat occurs in open marshes, though such marshes also have produced some excellent muck. Between the valuable muck lands of the hardwood forests and the worthless brown peat of the open marshes there is every degree of composition and every condition of decay and settlement. Similarly, as to the content of mineral matter, there is every degree of variation, from the peat of the Everglades and of Minnesota wire-grass marshes, which on burning leave only 3% of ash, to muck areas so impregnated with silt by the frequent overflow of muddy water that the soil on burning loses only one-third or one-fourth of its weight. Naturally, the mineral content affects directly the possibility of subsidence. Mr. Okey's paper would have been of still greater value if he had recorded the percentage of ash in the various classes of muck investigated.

Perhaps no other soils in the United States are so imperfectly known, or vary so much between the highest agricultural value and complete worthlessness, and probably no other soils have been so exploited by promoters—sales of worthless peat being made on the reputation of valuable black muck. A continuation of this investigation, as suggested by Mr. Okey, would add greatly to the knowledge neces-

sary for their economical development. The most significant inference from the paper is that, in consideration of the extremely flat gradients which sometimes are all that are possible, and the comparatively narrow range of fluctuation in ground-water level suitable for successful muck land agriculture in the South, all efforts to secure satisfactory gravity drainage in certain notable undertakings for the reclamation of peat and muck lands must be failures, and that the only hopeful prospect in these particular cases is for drainage by pumping. Mr. Morgan.

ORRIN RANDOLPH,* ASSOC. M. AM. SOC. C. E. (by letter).†—Mr. Okey has furnished information on a subject of prime importance in connection with the drainage of muck lands, and his investigations, when completed, should make possible a more accurate forecast of the behavior of these soils after the water-table has been lowered and farming operations have been conducted. Mr. Randolph.

The author has collected some of his data in Louisiana and the remainder in Florida, and the soil samples which have been examined have differed very materially, as is shown by the weights that were found. This wide difference in the nature of the material examined has produced results from which it becomes difficult to draw general conclusions.

However, the observations made on the unfarmed saw grass muck lands of Florida constitute a set of experiments on soil which is comparatively uniform in character and from which certain deductions may be made.

By making use of the actual weights shown on the five Florida profiles of unfarmed saw grass muck, and reducing the weight of each foot in depth to its percentage of the total weight of the sample from which it is taken, a set of results is obtained as shown in Table 2.

An examination of Table 2, in connection with the graphic illustration, Fig. 19, shows the percentages of weights which were found.

In these five cases the average lowering of the water-table below the surface was 2.3 ft. The average subsidence was 1.6 ft., and the time the land had been drained, although not given in every case, would probably average about 2 years.

It would appear from the results found, and from observations of the writer, that the subsidence of saw grass muck, due to a lowering of the water-table, takes place somewhat uniformly over the entire depth of muck, and that it is not confined to that portion of the muck which stands above the water-table. It also appears that the depth of subsidence due to this cause is dependent on the depth to which the water-table has been lowered, as well as the time, within certain limits, that has elapsed after the drainage operations have become effective.

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* Lake Worth, Fla.

† Received by the Secretary, November 2d, 1917.

Mr. Randolph. The author did not explain his method of obtaining and ascertaining the weight of the samples, and it is to be hoped that this information will be given, so that, in case independent observations are made, a basis of comparison of results may be had.

TABLE 2.—PERCENTAGE, IN WEIGHT, OF TOTAL SAMPLE.

Location.	1st foot.	2d foot.	3d foot.	4th foot.	5th foot.	6th foot.	7th foot.	8th foot.	9th foot.	Average weight per cubic foot of sample.
Fellsmere District Lat. Q. North...	12.8	9.3	10.2	12.1	13.4	12.4	9.5	9.9	10.4	6.33
Fellsmere District Lat. Q. South...	9.9	9.0	9.2	10.5	10.2	14.4	12.8	11.1	12.9	5.56
Fellsmere District Lat. S. North....	9.3	9.5	10.9	10.9	14.7	14.0	11.0	10.5	9.2	5.09
Indian River District............	6.3	8.1	9.0	9.7	11.9	13.1	11.9	13.3	16.7	8.86
Upper Everglades District, Okeelanta............	12.1	8.9	9.8	11.3	11.7	13.1	10.0	9.4	13.7	6.19
Totals.............	50.4	44.8	49.1	54.5	61.9	67.0	55.2	54.2	62.9	32.03
Averages..........	10.08	8.96	9.82	10.90	12.38	13.40	11.04	10.84	12.58	6.406

AVERAGE PERCENTAGE, IN WEIGHT, OF FIVE FLORIDA SAMPLES

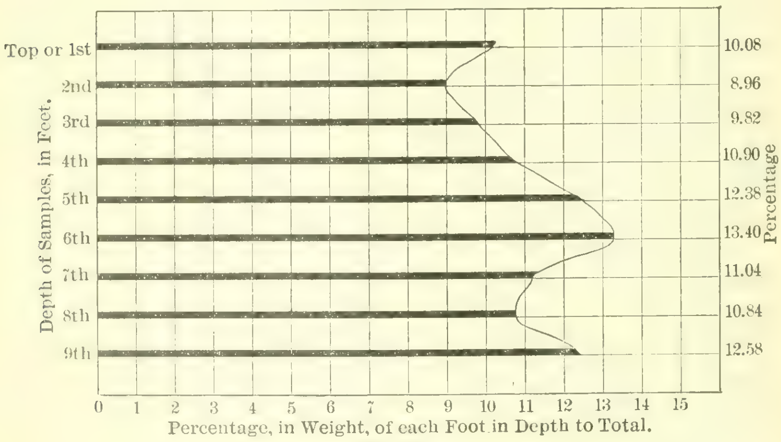

FIG. 19.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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PAPERS AND DISCUSSIONS

DISCUSSION ON PROGRESS REPORT OF THE SPECIAL COMMITTEE TO CODIFY PRESENT PRACTICE ON THE BEARING VALUE OF SOILS FOR FOUNDATIONS*

By William Cain, M. Am. Soc. C. E.

William Cain,† M. Am. Soc. C. E. (by letter).‡—The stress-strain curves, Figs. 2, 3, and 4, for differing densities of the Ottawa sand experimented with, show the importance of density determinations in studying the properties of earths, and that, "large variations in the physical properties accompany slight changes in density of the mass." The gauge of Fig. 1 (*a*) is evidently a very perfect one, so that the pressures recorded by it can be relied on. Mr. Cain.

When this gauge was placed in the center of the base of the cylinder, Fig. 1 (*a*), and 1 in. of sand was placed on it and pressure applied through the piston, the pressure on the diaphragm at the center was found to be in excess of the hydrostatic pressure by 25%; whereas, with 5 in. of sand on the base, the pressure indicated by the gauge was less than the hydrostatic pressure. These results are shown graphically in Fig. 6. They indicate that the pressure of the sand on the base was not uniformly distributed over it.

In the case of the 1 in. of sand on the base, evidently but little of the piston load was carried by the side walls by friction of the earth on the metal of the cylindrical surface, so that nearly all the piston pressure was transmitted to the base; but the experiments indicate that the intensity of pressure on the base was not uniform, it being greater at

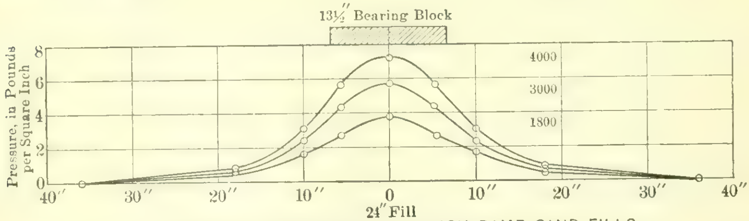
* Presented to the Annual Meeting, January 17th, 1917.

† Chapel Hill, N. C.

‡ Received by the Secretary, October 15th, 1917.

Mr. Cain. the center and, thus, less at the sides, than the average, or the piston pressure divided by the area of the base—the hydrostatic pressure.

In the case where the base was covered with 5 in. of sand—to judge from the independent experiments of various authors—much of the piston load was held up by the wall friction (earth on metal); so that, though the intensity of the pressure on the base at the center, as found by the gauge there, was less than the average, it seems reasonable, as before, to assume that this intensity decreases from the center to the sides. In this case, only a part of the piston pressure is transmitted to the base, the remainder being carried, by friction, to the side-walls. It is to be expected, therefore, that the average intensity of pressure on the base should be less than the total piston pressure divided by the area of the base; and, with the side-walls carrying enough of the load, it is easily understood why the gauge at the center of the base indicated an intensity there (the maximum) less than the last-mentioned average.



PRESSURE DISTRIBUTION THROUGH DAMP SAND FILLS

Fig. 17.

Considerable light is thrown on this subject by the experiments of A. T. Goldbeck,* Assoc. M. Am. Soc. C. E., where sand of a uniform depth of from 6 in. to 5 ft. was subjected to a load applied to a flat circular bearing block 8 or 13½ in. in diameter. The sand was placed in a bin 7½ ft. square when the depth of sand was 2 ft. or less, the bin being larger for greater depths of sand. The bearing block was placed on top of the sand at the center of the bin.

The vertical unit pressure at a number of points at the bottom of the sand fill was ascertained by the aid of a number of small cells placed there, the pressure being found by equilibrating the air pressure in a cell with the soil pressure exerted on a movable part of the cell. The air pressure in the pipe leading to the cell was increased slowly until equilibrium was attained, the exact instant being detected by the breaking of an electric contact within the cell, at which instant the air pressure was measured with a sensitive gauge.

The diagram, Fig. 17, shows the variation in the pressure at various distances, in inches, from the center of the block, the ordinates indi-

* "The Distribution of Pressures Through Earth Fills", *Proceedings*, Am. Soc. for Testing Materials, 1917. Also, see *Engineering News-Record*, July 19th, 1917.

cating the difference between the pressures for block loaded and for sand alone, the pressures being in pounds per square inch. The upper, middle, and lower curves, correspond to loads on the block of 4 000, 3 000, and 1 800 lb., respectively—all, for a depth of 24 in. of the damp sand fill, having from 3 to 5% of moisture. The curves are typical for depths of sand varying from 6 in. to 5 ft., but they become flatter and extend farther, in proportion to the height, as the depth of the sand increases. Mr. Cain

Evidently, the volume generated by revolving one of the curves about the vertical axis through 0 equals the reaction of the base, and this was found practically to equal the load applied, thus giving a desirable check on the results. The part of the volume considered, directly under the block, gives the load transmitted to the part of the base directly under the block, the remainder of the load being transmitted, through friction of the earth particles, outside this area. It is seen from Fig. 17, that a large part of the load is distributed over the annular area outside of that directly under the block.

It will be observed from Fig. 17 that the unit pressure on the base is a maximum under the center of the bearing block, and that it decreases as the periphery of the block is approached, according to the ordinates of some curve. The decrease is much more marked for the 6 and 12-in. fills. In fact, for the 6-in. fill, with an 8-in. bearing block, a diagram in Mr. Goldbeck's paper shows, for a load on the block of 1 400 lb., a unit pressure at 0 of 26.5 lb. per sq. in., whereas, at a point vertically under the edge of the block, the pressure on the base was only 9.7 lb. per sq. in.

The conditions, for the sand in the cylinders, Fig. 1 (*a*) and (*b*), of the Sub-Committee's report, are not the same as for the sand fill, since the sand under the piston in the cylinders is bounded by rigid metal walls, whereas that immediately under the bearing blocks, in the case of the sand fill, is bounded by non-rigid sand walls, transmitting lateral pressure, and it is well known that very slight displacements in a sand fill modify materially the state of stress at and near the points of displacement. In either case, however, the "walls" carry a certain portion of the load through friction, and there is sufficient similarity in the two cases to lead one to believe that the unit pressure on the base of the metal cylinder decreases, from a maximum at the center to a minimum at the containing wall, though the law of this decrease may not be exactly the same as for the portion of the sand fill vertically under the bearing block.

It will be observed from Fig. 17 that the unit stress on the base varies according to the ordinates of some curve, which, for the upper portion, nearly to the inflection point, resembles a parabola, and it will be taken as such in the investigation, given subsequently, in connection with Fig. 19, referring to the rotary cylinder.

Mr. Cain. The cylinders of Fig. 1 are in reality bins, when filled with sand to a depth of 5 in. or more, and a modification of the bin theory due to the piston pressure is applicable; but a simpler theory will suffice to estimate, approximately, the proportion of the piston load carried by the cylindrical walls for the small depth of sand considered. Thus, if P is the piston pressure, the diameter of the piston being 12 in., the unit vertical pressure is $\frac{P}{\pi \times 6^2}$, and, if we take the lateral unit pressure of the sand as one-third of this, the pressure $= \frac{1}{3}\,\frac{P}{36\,\pi}$. This, for a depth of sand of 5 in., acts on an area, $2\,\pi \times 6 \times 5 = 60\,\pi$, so that the total lateral pressure on the cylindrical wall is the product, or $\frac{5}{9}\,P$. If the coefficient of friction of sand on iron is taken arbitrarily at 0.45, the total load carried by the cylindrical walls is $0.45 \times \frac{5}{9}\,P = 0.25\,P$.

In experiments on a large wooden bin, filled with grain, Bovey found that the vertical unit pressures on the base diminished from the center to the side of the bin.* If, in the case of the cylinder, Fig. 1 (*a*), by some device, the wall is made perfectly smooth, then it cannot carry any of the piston load, and thus the whole load is transmitted to the base, which may experience a nearly uniform distribution of stress. For the rotary cylinder, Fig. 1 (*b*), the walls and base, of necessity, must be left rough to fix the sand in position and prevent it from moving with the rotary disk; so that, for this apparatus, a uniform vertical component of stress on the base is probably never realized.

The subject is a vital one for computations pertaining to either cylinder, for, in Fig. 1 (*a*), whether the gauge is placed at the bottom or on the walls of the cylinder, it is necessary to be able to compute, first of all, the intensity of the vertical component of stress at the center of the gauge; and, similarly, for the rotary cylinder, Fig. 1 (*b*), the vertical component of stress at any point of the base must be capable of estimation. In fact, if this component is not uniform over the rotary disk, the formulas deduced by the writer† are inapplicable, as was expressly stated in the paper referred to.

The results of the experiments on sand with the rotary cup disks, Fig. 9, are distinctly disappointing, because, as the authors state essentially, almost any pressure and coefficient of friction can be found, between the limits shown on the diagram, for "incipient" and "actual" motion—the less the motion, the less the friction.

* As shown in Fig. 177 of Ketchum's "Walls and Bins."

† "Cohesion in Earth", *Transactions*, Am. Soc. C. E., Vol. LXXX (1916), pp. 1336-1337.

It would seem then, for a given motion at the edge of the rotary disk, as the displacement decreases uniformly to zero at the center, that the coefficient of friction should decrease, from a maximum value at the periphery of the disk, gradually toward zero as the center is approached. It is true that the formula used in computing the coefficient of friction is inapplicable if this law holds, still the results lead to comparative values that suggest the law—the less the motion, the less the friction. It may be that the coefficient of cohesion also diminishes toward the center. Mr. Cain.

It is seen from the diagram, Fig. 9, that the coefficients of friction for "incipient" motion do not average as much as 0.05, and for "actual" motion, the average is less than 0.20, whereas we should expect, for the "dry sand, moderately compacted", about the usual value, $\frac{2}{3} = 0.67$. It may be objected that this value has been obtained by noting simply the tangent of the angle of repose for dry sand, but experiments by Leygue and Bell will be cited where the coefficient was found by dragging sand in a box or cylinder with no bottom, over sand in a box or cylinder. In the experiments of Leygue,* the normal pressures were very small—only from 7 to 40 lb. per sq. ft. They were as follows:

Dry sand.........$f = 0.70$, $\phi = 35°$, $k = 1.47$ lb. per sq. ft.
Wet sand.........$f = 0.85$, $\phi = 40° 22'$, $k = 8.28$ " " " "
Very wet sand...$f = 1.70$, $\phi = 59° 30'$, $k = 6.36$ " " " "

In a paper by Arthur Langtry Bell on "The Lateral Pressure and Resistance of Clay and the Supporting Power of Clay Foundations",† it is stated that when perfectly dry sand was "well rammed" in the cylinder, it was found that k was practically zero and $\phi = 30°$, the angle of repose; but, if the sand was merely poured in the cylinder and shaken, although k was again zero, the angle of friction was much less than the angle of repose. For "wet sand grabbed from a monolith well", $k = 0.3$ ton per sq. ft. and $\phi = 31$ degrees.

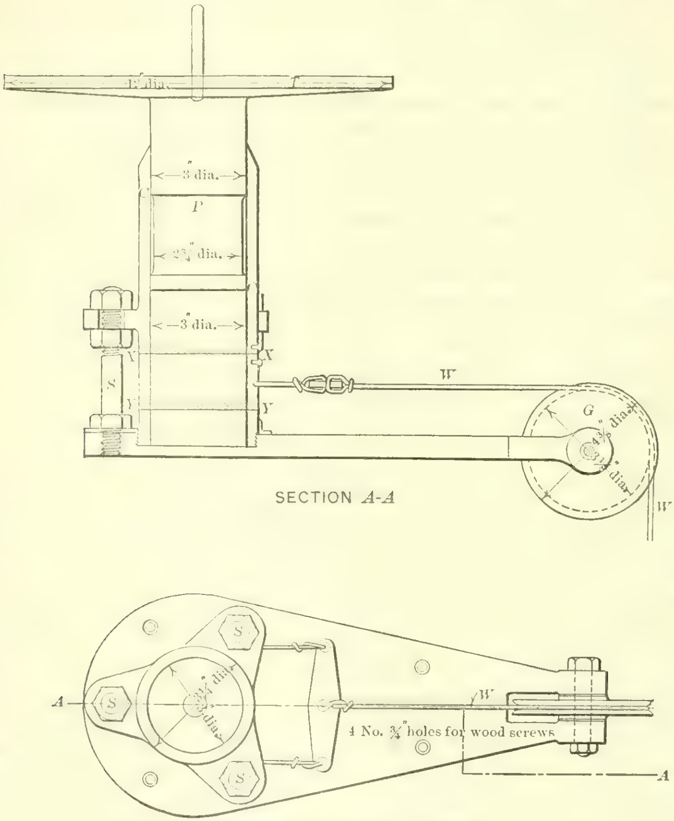
Bell's apparatus is shown in Fig. 18. It consists of a solid cylindrical cast-iron plunger, P, with machined faces, working inside a brass cylinder, C. The brass cylinder is cut through horizontally along the two parallel planes, XX and YY, the cut surfaces being perfectly smooth and polished. The ring, therefore, is free to slide in a horizontal direction. By means of the three screws with nuts, S, the pressure on the horizontal metal sliding surfaces can be adjusted as desired and the pull required to withdraw the empty ring can be ascertained and subtracted from W, the pull required to cause sliding along XX and YY when the cylinder is partly filled with clay and a load applied through the plunger. The normal unit loads on a surface, as XX, were plotted as

* *Transactions*, Am. Soc. C. E., Vol. LXXX (1916), p. 1319.

† *Minutes of Proceedings*, Inst. C. E., Vol. CXCIX, 1914–15, Part I.

Mr. Cain. abscissas and the values of the shears on the clay surface, XX, corresponding, as ordinates, and the usual straight line drawn, from which k and $f = \tan. \phi$, could be ascertained.*

It would seem that this apparatus was more especially adapted to clay, as it prevents flow and but little of the weight on the plunger is carried to the cylindrical walls, as the friction of clay on smooth brass



PLAN: PLUNGER REMOVED

Fig. 18.

is practically nil. This is not true for ordinary earth or sand. The problem, in any case, is simply to drag earth over earth, subjected to varying definite loads and to measure the shearing forces required and also the area of the surface of shear. The loads should be placed on

* Some of the results are given in *Transactions*, Am. Soc. C. E., Vol. LXX, p. 1322. The remarks of Dr. Unwin, p. 1322, and of Mr. Goodrich, p. 1330, are pertinent in this connection.

top of the bottomless box containing the earth above the surface of shear, and not on the earth; then the load on this surface is exactly known. This is not the case with the cylinder, Fig. 18, where the load is placed directly on the earth, and thus part of the plunger load is carried by friction of earth on metal to the cylindrical walls. Mr. Cain.

It seems desirable to have tests of clay made with this or a similar apparatus, but the results of the tests would be more convincing if the diameter of the cylinder was larger, say, 12 in., and if the load on the plunger was measured on the beam of a testing machine. From the results of the experiments just recorded, it is seen that, for clean, dry sand, the angle of friction is very near the angle of repose, which, generally, equals or exceeds 30 degrees. Thus, the friction coefficients shown in Fig. 9 are much too low, and the results have only a comparative value.

The same remarks apply to the results exhibited graphically in Figs. 13-15 of the Sub-Committee's report. In these tests, varying percentages of water were added to the sand, and the computations gave values of ϕ varying from 9° 40′ to 16° 40′, and values of k, from 0 to 8 lb. per sq. in., whereas we should naturally expect ϕ to exceed 30° and k to have a much lower value than the maximum stated. The discrepancies are partly due to the fact that, in the formula used, (1) the walls were not supposed to carry any of the piston load, though its existence was fully realized, (2) the normal unit pressure on the rotary disk was assumed to be uniform and equal to the mean piston pressure, and (3) the coefficient of friction was assumed to be constant over the surface of the disk.

To form some idea of the relative influence of each of these hypotheses, it will be arbitrarily assumed: (1) that the cylindrical walls carry one-fourth of the piston pressure, (2) that the normal pressures on the base vary as the ordinates of a parabola, and (3) that the coefficient of friction on the disk is zero at its center and uniformly increases to a maximum at the circumference of the disk. From what precedes, it is evident that the load carried by the walls and the law of variation of normal pressure on the base can only be ascertained by experiment, though it is plain that the proportionate load carried by the walls will vary with the depth of sand, and the coefficients of friction of sand on sand and sand on metal, and thus can vary within wide limits.

To judge from the experiments on bins and the sand fill, Assumption (2) is reasonable, and Assumption (3) finds some support from the first set of experiments with the rotary cylinder. All the assumptions are plausible, so far as our present knowledge extends, but it must be distinctly understood that the formulas deduced below are only to be used for comparative purposes.

In connection with Figs. 13-15, the Sub-Committee states that the normal pressure on the base (marked p_n in the figures) represents the

Mr. Cain. mean pressure on the large piston, or $\frac{P}{A}$, if P is the piston load and A is the area of the base of the piston. Conceive, then, the abscissas in the figures to be the values of $\frac{P}{A}$, and let p_n represent the revised value of the normal pressure on the base, as computed below.

Take, as an illustration, the case where 5% of water was added to the sand, Fig. 13, where it was found that $k = 0$ and that the straight line drawn made an angle of 16° 40′ with the horizontal; whence, if $\frac{P}{A} = 100$ lb. per sq. in., $q = 100$ tan. 16° 40′ = 29.97 lb. per sq. in.

Case (*a*).—Let p_n, f, and k be regarded as constant, and assume that the cylindrical wall carries $\frac{1}{4}P$; whence, as the vertical pressure on the base is now supposed to be uniformly distributed and equal to $p_n = \frac{3}{4}\frac{P}{A}$, we have, corresponding to $\frac{P}{A} = 100$, $p_n = 75$, and $q = 29.97$, as given above.

If ϕ is the angle of friction of sand on sand, then tan. $\phi = \frac{29.97}{75} = 0.399$, and $\phi = 21° 45'$.

Case (*b*).—Let f and k be constant, but suppose p_n to vary as the ordinates of a parabola, Fig. 19, and that, as before, the cylindrical wall carries one-fourth of the piston load.

The radius of the rotating disk $= r = 3.09$ in., and the radius of the base of the piston $= R = 6$ in.

Let the area of the disk $= a = \pi r^2$, and the area of the piston $= A = \pi R^2$; whence, $\frac{a}{A} = 0.265$.

The vertical unit pressure, in pounds per square inch, at the distance, ρ, from the center, O, will be called p_n, and the values of p_n at $\rho = o$, $\rho = r$, $\rho = R$ will be designated p_0, p_1, and p_2, respectively, as marked on the figure.

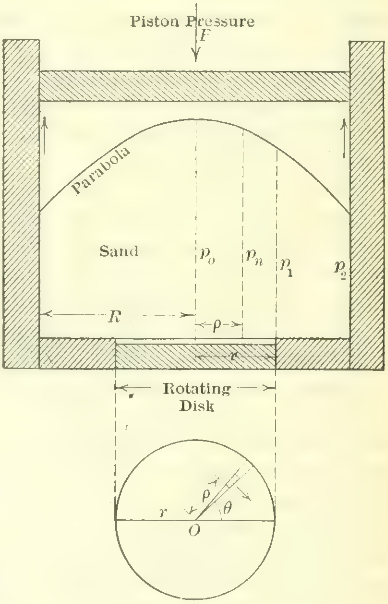


FIG. 19.

When the rotary disk is given a slight rotary motion, the sum of the moments of the forces resisting sliding $= M$ also equals the moment of the external forces.

The equation of the parabola is, Mr. Cain.

$$p_n = p_o - \frac{p_0 - p_2}{R^2} \rho^2.$$

By Coulomb's law, the resistance to shear, in pounds per square inch, at a point (ρ, θ) is, $f\, p_n + k$, when $f = \tan. \phi$, is the coefficient of friction and k is the coefficient of cohesion. Whence, the resistance to sliding on the elementary area, $\rho\, d\, \rho\, d\, \theta$, is $(f\, p_n + k)\, \rho\, d\, \rho\, d\, \theta$, and its moment about O is ρ times this expression. On substituting the value of p_n above, integrating and reducing, we find,

$$M = \int_0^{2\pi} \int_0^r (f\, p_n + k)\, \rho^2\, d\, \rho\, d\, \theta$$

$$= [(0.841\, p_0 + 0.159\, p_2)\, f + k] \frac{2}{3}\, r\, a;$$

whence,

$$q = \frac{M}{\frac{2}{3}\, r\, a} = (0.841\, p_0 + 0.159\, p_2)\, f + k.$$

As a check: When p_n is constant, $p_0 = p_2$, and the right member reduces to, $p_n\, f + k$, as hitherto found.*

As p_n is in pounds per square inch, the total load on the base of the cylinder, $\frac{3}{4} P$, is given by the volume of the solid of revolution generated by revolving the surface bounded by the horizontal axis through O, the ordinates, p_0 and p_2, and the parabola, about the vertical axis through O. The volume equals that of a cylinder of height, p_2, area of base, A, plus that of the paraboloid of revolution surmounting it; hence, as the volume of the paraboloid is one-half that of the circumscribing cylinder, it is at once found that,

$$\frac{1}{2}\, (p_0 + p_2)\, A = \frac{3}{4}\, P.$$

From lack of experimental knowledge as to the ratio of p_2 to p_0, assume, arbitrarily, $p_2 = \frac{2}{3}\, p_0$, whence, from the last equation, $p_0 = 0.9\, \frac{P}{A}$. On substituting these values in the preceding equation, we obtain,

$$q = 0.853\, \left(\frac{P}{A}\right) \tan.\, \phi + k.$$

As P varies, if the successive values of $\left(0.853\, \frac{P}{A}\right)$ and q are laid off as abscissas and ordinates, the equation represents that of a straight line making the angle, ϕ, with the axis of abscissas and having an

* "Cohesion in Earth," *Transactions*, Am. Soc. C. E., Vol. LXXX (1916), p. 1336.

Mr. Cain. intercept on the q axis equal to k.* Using the simultaneous values, $q = 29.97$, $\frac{P}{A} = 100$, as g ven above for the case $k = 0$ and 5% water, Fig. 13, we have,

$$\tan. \phi = \frac{q}{0.853 \frac{P}{A}} = \frac{29.97}{85.3} = 0.351\,;$$

therefore

$$\phi = 19^\circ\ 20'.$$

Hence, comparing this value of ϕ with that of Case (a), it is seen that the influence of the supposed parabolic variation in p_n is to diminish ϕ.

Case (c).—Let k, only, be regarded as constant, and suppose p_n, as in Case (b), to vary according to the ordinates of a parabola, and f to vary uniformly from zero at O to a maximum, f_1, at the edge of the disk, so that,

$$f \text{ at } (\rho, \theta) = \frac{f_1}{r}\,\rho.$$

Also, as in Cases (a) and (b), regard $\frac{1}{4}\,P$ as carried by friction by the cylindrical walls.

On substituting this value of f and the previous value for p_n in the general expression given above for M, it is found, after reduction, that,

$$q = \frac{M}{\frac{2}{3}\,r\,a} = [(0.617\,p_0 + 0.132\,p_2)\,f_1 + k].$$

The value of q here, as hitherto, is to be regarded as simply the value $M \div \frac{2}{3}\,r\,a$, using the value of M as given by experiment.

Assuming, as in Case (b), $p_2 = \frac{2}{3}\,p_0$, we derive,

$$q = 0.634\,\left(\frac{P}{A}\right)\,f_1 + k.$$

For the line of Fig. 13 corresponding to 5% of water and $k = 0$,

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*The last equation can also be conveniently plotted by computing, $q' = \frac{q}{0.853} = \frac{M}{0.853 \times \frac{2}{3}\,r\,a} = \left(\frac{P}{A}\right) \tan. \phi + k$, and laying off the corresponding values of $\left(\frac{P}{A}\right)$ an q', as abcissas and ordinates. It will be seen that the final equations for all the cases can be plotted as straight lines.

we have, as before, $q = 29.97$, corresponding to $\frac{P}{A} = 100$; whence, from the last equation, Mr. Cain.

$$f_1 = \text{tan. } \phi = \frac{29.97}{63.4} = 0.473;$$

therefore

$$\phi = 25° \ 19'.$$

The third supposition thus leads to a greater ϕ than that given in Case (a), and a more probable value. The changes in the computed value of ϕ, starting with the initial value, 16° 40′, will be seen from the following values of ϕ, corresponding to Cases (a), (b), and (c): 21° 45′, 19° 20′, and 25° 19′.

If we take the lower line of Fig. 13, which makes only the angle 10° 10′ with the p_n axis and corresponds to perfectly dry sand, with $k = 0$, the values of ϕ for Cases (a), (b), and (c), are, 13° 27′, 11° 52′, and 15° 48′.

The values of ϕ for all the other cases of Figs. 13-15 will lie between the extremes for the two cases given, and thus are all too small, for, from the independent experiments cited above, we should expect ϕ to exceed 30 degrees. Evidently, some other influence has been experienced than the friction on the side-walls and the variation of p_n and f. From the results pertaining to Fig. 9 and the remarks of the Sub-Committee pertaining thereto, it seems that the full friction of sand on sand was not exerted because of the very small incipient motion. It is not stated in the report whether Figs. 13-15 refer to "incipient" or "actual motion." From Fig. 9, for actual motion at the rack of 0.01 to 0.10 in., the average friction for incipient motion was about trebled; and it would be interesting to know, if the actual motion is still further increased, whether there would be an increase in the computed friction, and also the result, if the rack is moved at a uniform rate to avoid acceleration.

From Fig. 9, it is likewise observed that, on mixing plaster of Paris with the sand, to bind together the grains of sand at the surface of the disk, the coefficient of friction was more than doubled. This is very significant, and would indicate that when the loose grains of sand are mixed with increasing quantities of clay—thus forming a series of ordinary earths—the true coefficients of friction and cohesion in earth would be more nearly realized.

It has been assumed, in the case of the rotary cylinders, that their inner surfaces and the base outside the rotary disk were sufficiently rough to fix the sand in position near them. If this is not realized, the results would be invalidated. In fact, if these surfaces were perfectly smooth, the rotation of the disk would simply drag around the sand in the cylinder. It is possible, on account of the smoothness

Mr. Cain. of these surfaces, that some small motion of this kind is realized, particularly for "incipient motion", in which case, the computations made on the basis of a fixed cylinder of sand would be invalidated. These last remarks are especially pertinent in the case of clay, when experimented on in the rotary cylinder; for the coefficient of clay on metal is, perhaps, very small, so that it is possible that not enough friction exists to "fix" the cylinder of clay and prevent its rotation.

The writer, in what precedes, has been endeavoring to account for those low values of ϕ shown in Figs. 13-15, and, incidentally, has shown something of the very complex state of stress of the earth in the rotary cylinder when subjected to pressure. It seems to be impossible to effect a solution in the present state of our knowledge. This is disappointing, as the apparatus, at first sight, promised useful results. There is a complex state of stress when sand is subjected to pressure in a non-rotary cylinder, though to a less extent than in the former case. On account of this complexity, the writer has never been very hopeful of attaining practical results with any apparatus where the earth is confined in a small space, and thinks that many published results, derived by aid of such an apparatus, are very misleading.

The experiments relating to "conjugate pressures", made by aid of the apparatus shown in Fig. 16, the results being exhibited graphically in Fig. 8, are interesting. As the Sub-Committee states, referring to the relation,

$$\frac{p_b}{p_a} = \left(\frac{1 - \sin. \phi}{1 + \sin. \phi}\right)^2,$$

"the equation defines the mode of equilibrium at a definite point in a cohesionless material when the particles exert their maximum frictional resistances." Hence, for the theory to be exactly realized, both piston and gauge would have to be of infinitesimal dimensions, with their centers only a minute distance apart. Mechanical difficulties limit the least practicable distance between these centers to 4 in., so that it is probable that the lateral thrust in the sand due to the piston pressure is changed in amount and direction in traversing the 4 in., so that the theory is not exactly realized. It would be less exact if the distance between centers was further increased. Although the mean values of ϕ computed from the formula are doubtless not the true ones, still, the comparative results are valuable for values of p_a varying from 0 to 300 lb. per sq. in., and they seem to indicate, as stated, that internal "friction is a function of the pressure and the density." The experiments by Bell, cited previously, relating to sliding of sand over sand under varying pressures, are in line with this statement, for when the sand was "well rammed" in the cylinder, the derived value of ϕ was much greater than when the sand "was merely poured in the

cylinder and shaken." Similar experiments by MM. Jacquinot and Frontard*, on earth taken from a dam, indicated but little change in ϕ from ramming or puddling, but it was stated that puddling and ramming could more than double the coefficient of cohesion. Mr. Cain.

The Sub-Committee's report is necessarily a preliminary one, and "the results of experimentation submitted at this time are to be considered tentative." The experimental work, of course, is of a high order, and is valuable as an introductory investigation, particularly as to the relative value of various methods of experimentation.

As the tests proceed, there will doubtless be a gradual elimination of the "unfit", and a "survival" of the best methods and apparatus for dealing with soil pressures under various conditions such as occur in practice.

* Referred to in *Transactions*, Am. Soc. C. E., Vol. LXXX (1916), p. 1321.

MEMOIRS OF DECEASED MEMBERS

NOTE.—Memoirs will be reproduced in the volumes of *Transactions*. Any information which will amplify the records as here printed, or correct any errors, should be forwarded to the Secretary prior to the final publication.

JOHN HATFIELD FRAZEE, Assoc. M. Am. Soc. C. E.*

DIED MAY 4TH, 1917.

John Hatfield Frazee, the son of Lawrence Fisher Frazee and Sarah Lee (Stark) Frazee, was born at South Amboy, N. J., on September 23d, 1867. His father, who was also born in New Jersey, was engaged in the railroad and transportation business, and, during the Civil War, had charge of transporting troops for the Federal Government. He devoted much of his time to various inventions, including a lifeboat, which was adopted by the Navy Department, and a canal-boat which was designed to avoid the destruction of banks by wave action; he also invented the collapsible gates commonly used on elevated railway and subway trains and on ferry-boats, and applied for and secured patents on various devices for improving transportation facilities.

His son, John Hatfield Frazee, received his engineering degree from New York University, from which he was graduated in June, 1886. Shortly after he entered the office of the late Charles B. Brush, M. Am. Soc. C. E., being assigned by Mr. Brush to general city work. Entering the service of the Pennsylvania Railroad during the summer of 1887, he served in the field party of E. F. Brooks, Engineer of Maintenance of Way, on the Princeton Draw-bridge, the new passenger line through Elizabeth, N. J., and on a number of bridge and grade changes, including the Jersey City elevated and terminal work. Mr. Frazee was then appointed Assistant Engineer in charge of the location and construction of the Jersey City elevated and terminal work, and, on its completion, had charge of the Harsimus Cove third-track improvement, including four bridges and various improvements in and about the Jersey meadows and river crossings. He was then given charge of the elevated work contemplated through Elizabeth, including the location and construction of abutments, piers, ribbed arch bridges, retaining walls, and about 5 000 ft. of double-track trestle.

In the early part of 1893, Mr. Frazee resigned from the Pennsylvania Railroad service to become Engineer of Construction on the Lancaster and Columbia Railroad, a 12-mile line of standard con-

* Memoir prepared by R. W. Creuzbaur, M. Am. Soc. C. E.

struction. Afterward he had charge of the location and construction of trolley lines in and about Hartford, Conn., and, subsequently, of the location, estimates, and construction of the Ivy Hill Storage Reservoir, for Newark, N. J. In May, 1894, he was appointed Chief Engineer of the Hartford Street Railway. This System was about to be changed from horse to electric power, and the work planned and carried out involved the reconstruction of existing lines, 20 miles of new tracks, trestles, bridge abutments, and power stations, all on the best standards.

In February, 1897, Mr. Frazee entered the service of the City of New York, and his record up to the time of his death shows great activity and capacity not only for thorough study of engineering projects and contracts, but also for many constructive changes in various branches of city work. He was first engaged in 1897 as Assistant to the writer on street and highway improvements in the Department of Public Works in the old City of New York. This work cost $3 000 000 during that year, and included the reconstruction of Park Avenue over the New York Central Railroad tracks, and many street changes and improvements. He was also specially detailed at this time on studies of electrolysis of underground pipes, etc. The late Edward P. North, M. Am. Soc. C. E., was Consulting Engineer of the Department, and did much, by his able professional leadership and active interest, to mould the efforts of the entire Engineering Corps toward high standards of ethics and efficiency.

On the consolidation of "Greater New York" in January, 1898, the Highway Bureau of Manhattan had jurisdiction over the whole city, and the engineers proceeded to standardize specifications for such work, Mr. Frazee having an important part in this difficult undertaking. Subsequently, and after the writer had left this branch of the city service, Mr. Frazee resigned to engage for some time in his former specialty of electric railway construction, as Resident Engineer of the Dallas (Tex.) Consolidated Electric Street Railway. He was afterward engaged on the location and designs for the New York Connecting Railway, including the Hell Gate Bridge project, on which, in its initial stages, he made recommendations for changes showing large economy in the estimates of cost.

In December, 1904, he accepted a position with the Department of Finance of New York City, reporting to the writer in the general supervision of engineering work. In the early part of the following year, he was duly appointed a First Assistant Engineer, and thereafter served in many important matters, reporting, as a technical adviser of Comptrollers Coler, Grout, Metz, and Prendergast, on many hundreds of projects and contracts. He was advanced in five promotions until 1914, when he was transferred to the new Bureau of Contract Supervision of the Board of Estimate, for which, at the time of his death,

he was handling many of the most important municipal engineering questions and contract works, including all the dual subway contract propositions for the city.

The power of the Comptroller of the City of New York is exceptionally broad and far-reaching, and the members of the small staff of engineers reporting to him were entrusted with the investigation of projects, estimates of cost, and plans of every variety of engineering work, and were held responsible for the supervision and proper execution of such work after funds had been appropriated by the Board of Estimate. Mr. Frazee had special ability for analytical study of these matters, and, having gone through a training in design and construction, his reports on the many projects and contract work placed in his charge were gauged by the stern necessities of good practice and economy, and were of great practical and money value to the City. His associates looked to him, in these dozen years of such special service, as an expert of much ability along these lines of work, and, above all, as a man of absolute honesty and perfect candor in his straightforward exposition of defects in work or design and his constructive recommendations on plans and methods involved in the propositions of the many departments and bureaus having authority to spend the money of the taxpayers.

His many adverse reports unavoidably reflected on the conduct of these public affairs by the Borough Presidents, Commissioners, and Engineers, including many of the distinguished members of the Profession associated with the New York Departments, but every engineer who met the logic of Mr. Frazee, in the reports on these railroad, bridge, dock, sewer, and other questions, in the works which the City has been called on by them to finance, had respect for his ability and impartiality in handling the responsible duties assigned to him.

Referring only briefly to the most important matters on which he made such original recommendations for changes in plans and estimates prepared by the various departmental engineers, and where he contested claims connected with the many contracts made the subject of litigation and handled by the Comptroller's engineers, his recommendation to the writer of the advantage in municipal works of utilizing what was called "percentage unit bidding", is noted. This method was put into effect in the Bureau of Sewers, of Brooklyn, in 1907, where, for 10 years, it has operated to promote competition and reduce unbalanced bids, litigation, and undue profit or loss to the contractors. Again, in 1905, through Mr. Frazee's personal investigations, there was disclosed what were found to be extensive and serious defects in sewer construction in Brooklyn, of such far-reaching importance that it necessitated in 1906 a re-organization of the Bureau of Sewers and the appointment of a new Chief Engineer recommended by Mr. Frazee.

By diligent investigation he also brought out many cases where City funds were afterward saved in carrying out sewer and water supply projects and in certain sewage disposal plants under construction, and took the initial steps which enabled the City to resist false claims successfully. These are small matters compared with the saving effected by his logical and stubborn resistance to applications for large appropriations for contracts of questionable utility to be loaded on the City Treasury. In spite of more or less bitter opposition from the officials favoring these appropriations, Mr. Frazee scored over and over again, and, directly through his personal efforts, saved many millions. Two of these cases may be cited, and in these the writer, not then associated with the City, co-operated with him: First, he defeated a makeshift reconstruction of the Queensboro' Bridge to accommodate subway trains as well as elevated and surface lines. In 1908, or shortly after the Quebec Bridge failure, the first steps toward investigation of the strain sheets of the Queensboro' Bridge were taken, and the resolution was written, which President Coler, of Brooklyn, put through the Board of Estimate, allowing Henry W. Hodge, M. Am. Soc. C. E., to be appointed to investigate this defective bridge in which the steel, bid for at a high price, had, by the juggling of politicians, been increased some 37% over the original weights on which its carrying capacity had first been estimated. Having followed this construction from the first foundation work, and having been connected with the litigation arising out of this matter in the Bridge Department, Mr. Frazee was especially alert, in 1914, on the scheme of Commissioner Kracke of the Department of Bridges, approved by the Chief Engineer of the Public Service Commission, to carry shortened trains of the Municipal Railways Company, running up Broadway and through 59th Street, over the Queensboro' Bridge as a temporary expedient. This involved cutting down the roadway from 52 to 26 ft., allowing no turnout for vehicles for a length of 3 700 ft., and required in operation, on account of the low factor of safety in the main members, the spacing of only six-car trains at 3 400 ft., and a block signal system limiting a single train only on the river span, on four tracks, and other complications, making operation necessary at 29 miles an hour to approach the capacity of the subway main line. Absolutely without support and encouragement, Mr. Frazee started opposition to the plan which had the approval of the Administration, already committed to the project endorsed by the Public Service Commission and its engineers. The writer was able, as a committee of one of the Citizens' Union, to point out these facts to intelligent members and, at his request, Mr. Hodge and Robert W. Quimby, President of the Chamber of Queens, were added to his committee. The former co-operated heartily in all that had been done and proposed, with the result that the Administration reversed itself, the bridge was let alone,

and the river tunnel proposed at 59th Street is now nearing completion.

The estimated cost of the defeated plan in first construction, alteration, and subsequent changes in the subway amounted to $5 000 000, and it was when the plans were near consummation to waste this amount that the ideas of Mr. Frazee were carried out and the plan was abandoned, with this saving.

Another case was Mr. Frazee's consistent opposition to the proposed filtration plant for the New Croton Aqueduct supply in the easterly basin of Jerome Park Reservoir, where a contract had been awarded for $5 500 000, needing only the approval of the Board of Estimate to become operative. Mr. Frazee had made various reports on water supply and filtration, and was familiar with the earliest history of the Jerome Park Reservoir and the scandalous methods through which the original contractor had been relieved of a losing contract on the plea that a filter plant would be necessary in the easterly basin. His reports on this proposition were elaborate and far-reaching, taking up the present and prospective value of the city property proposed to be used for the filtration work at Jerome Park, and the actual conditions in the water-shed, the failure to prevent contamination—even by pans—at the railroad trestles crossing the reservoirs, and forceful arguments showing the opportunities for better sanitary protection of the water-shed, reinforced as they were by the very low and receding death rate from typhoid in New York City, all of which resulted in his final success and the abandonment of what represented in carrying charges an investment of $30 000 000 by the City, and, in the several years which have elapsed since that date, the satisfactory quality of the water supply through improved methods has justified this action of the City authorities. In this case, also, though the present Mayor, who was Chairman of the Water Committee, with nearly every other member of the Board of Estimate, had approved filtration, he was so broad-minded as to halt the proceeding on the protest and to select a committee of ten engineers, including Mr. Frazee, to discuss these questions, and although the representative filtration experts of the country and the engineers of the Water Departments opposed every argument advanced, Mr. Frazee scored an absolute triumph, and nothing more has been heard of filtration.

Mr. Frazee was also selected at various times to serve on a board of engineers to report to the City authorities on difficult matters affecting designs and ways and means of constructing subways and rapid transit extensions. When considering the question of whether or not New York City should build pipe galleries in connection with its more important main lines of subways, he examined very carefully the questions of first cost, up-keep, and the off-setting savings in mutilation and maintenance of pavements, and favored pipe gallery construc-

tion on certain lines of the subway work. In 1910, he was selected by the writer, acting as chairman of a committee for the Mayor, Comptroller, and Aldermanic President, to revise the contract forms and specifications for the entire Tri-borough System, presented to the Board of Estimate by the Public Service Commission as ready for bids. Out of forty radical changes recommended by this committee, in which Mr. Frazee took a foremost part, twenty-eight were accepted by the Commission as recommended, and twelve were made the subject of compromise, Mayor Mitchel being quoted at that time as stating that the work of the engineering committee would save $10 000 000 in the cost of the subway system. Mr. Frazee originated most of these constructive recommendations.

With the passing away of John Frazee, the City of New York has lost an engineer of inestimable value who, in the writer's opinion, after thirty years of close observation, has done more good work along original lines of engineering reform than any other man in the service. His personal characteristics were such that, although generous and open-handed to a fault among his friends and confidants, he stood out, even when unsupported, with inflexible determination in every case where his employer, the great corporation of the City of New York, had any interest, and it is to be regretted that available records and opportunities are such that a more complete account of the accomplishments of this faithful and absolutely honest engineer—unafraid to carry out and attempt the accomplishment of his highest ideals in the face of much opposition—cannot be assembled and made a part of the permanent record of the City of New York as well as of the American Society of Civil Engineers.

Mr. Frazee was elected an Associate Member of the American Society of Civil Engineers on December 6th, 1899.

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The Reading Room of the Society is open from 9 A. M. to 10 P. M. every day, except Sundays, Fourth of July, Thanksgiving Day, and Christmas Day.

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TELEPHONE NUMBER........................4600 Vanderbilt.
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* Appointed Director, October 9th, 1917, to fill the vacancy caused by the resignation of Frank G. Jonah.

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AMERICAN SOCIETY OF CIVIL ENGINEERS

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MINUTES OF MEETINGS
OF THE SOCIETY

November 21st, 1917.—The meeting was called to order at 8.30 P. M.; Director L. D. Rights in the chair; Chas. Warren Hunt, Secretary; and present, also, 176 members and 22 guests.

Two papers entitled "The Hell Gate Arch Bridge and Approaches of the New York Connecting Railroad Over the East River in New York City", by O. H. Ammann, M. Am. Soc. C. E., and "Stress Measurements on the Hell Gate Arch Bridge", by D. B. Steinman, Assoc. M. Am. Soc. C. E., were presented by the authors and illustrated with lantern slides.

The papers were discussed by Messrs. Gustav Lindenthal, C. E. Fowler, L. S. Moisseiff, H. H. Quimby, and I. Delson.

The Secretary announced that he had received written discussions from Messrs. Henry B. Seaman, W. H. Breithaupt, H. J. Bingham Powell, J. A. L. Waddell, F. H. Frankland, L. A. Waterbury, and F. D. Hughes. As the meeting had already lasted nearly three hours, and midnight was approaching, they could not be presented.

The Secretary announced the following death:

William Stone Johnson, of Boston, Mass., elected Associate Member, June 6th, 1894; Member, May 1st, 1906; died November, 1917.

Adjourned.

December 5th, 1917.—The meeting was called to order at 8.30 P. M. The meeting was held in the general office of the Society owing to the fact that all available meeting rooms in the Engineering Societies Building were occupied by the American Society of Mechanical Engineers. President George H. Pegram was in the chair; Chas. Warren Hunt, Secretary; and present, also, 110 members and 26 guests.

The minutes of the meetings of October 17th and November 7th, 1917, were approved as printed in *Proceedings* for November, 1917.

Chas. Warren Hunt, M. Am. Soc. C. E., addressed the meeting on "The Activities of the American Society of Civil Engineers During the Past Twenty-five Years."

On motion, duly seconded, it was

"*Resolved:* That Dr. Hunt's address be published in the *Proceedings* and *Transactions* of the Society, and that a vote of thanks be given to him not only for his services during the period covered, but for his valuable and interesting paper."

Messrs. E. Gybbon Spilsbury, C. E. Fowler, W. J. Boucher, T. Kennard Thomson, and John Bogart also spoke.

Robert Ridgway, M. Am. Soc. C. E., presented a letter from the Military Engineering Committee of New York in regard to the need of the Ordnance Department for 25 000 skilled machinists, men of mechanical trades, for construction of depots and machine shops in France for the necessary repair of ordnance. He stated that the assistance of the Military Engineering Committee in this State had been requested, and that it was desired, to raise 2 000 men in and about New York City, and that a recruiting office had been established. He asked the aid of all members of the Society in securing the men required from the mechanical trades.

The Secretary announced the election of the following candidates on November 27th, 1917:

As Members

Alexander Clarke Clogher, New York City
Wilbur Ward Davis, Melrose, Mass.
Robert Paul Durham, Chicago, Ill.

George Stovall Edmondstone, Portland, Ore.
Woolsey Finnell, Tuscaloosa, Ala.
Ezra Clark Garlow, Alliance, Ohio
Paul Emile Mercier, Montreal, Que., Canada
George Edward Rodman, New York City
Solon Jones Stone, Garden City, N. Y.
Carl Voetsch, Utica, N. Y.

As Associate Members

Robert Louis Acker, Billings, Mont.
Vincent Richardson Andrus, Kansas City, Mo.
Richard McLean Barker, Lawrenceburg, Tenn.
Francis Bates, Los Angeles, Cal.
Edmund Bulkley Besselievre, Bayonne, N. J.
George Leslie Bilderbeck, Groton, Conn.
Thomas Francis Bowe, Hackensack, N. J.
Herbert Leslie Boynton, Haverhill, Mass.
Edward Sedley Bres, New Orleans, La.
William Edward Brown, San Domingo, Dominican Republic
Lee Marshall Bush, Burlingame, Kans.
William Parker Butler, Nashville, Tenn.
Thomas Francis Campbell, Providence, R. I.
Frank Wilson Chappell, Dallas, Tex.
Robert Carr Churchill, Lester, Pa.
James Hopkins Clark, Calipatria, Cal.
George Cottingham, Jr., Carrington, N. Dak.
John Adam Crowley, Lynbrook, N. Y.
George Clapp Danforth, Gardiner, Me.
Marino Diaz Quinones, Havana, Cuba
Walter Edward Dickerson, Brownwood, Tex.
John Russell Ellis, Jefferson City, Mo.
William Lee Fewsmith, Glen Ridge, N. J.
Raymond Hanson Findley, Omaha, Nebr.
John Brandon Franks, Leavenworth, Kans.
Grant Friel, Los Angeles, Cal.
Charles Elijah Galt, St. Louis, Mo.
Ray Stuart Gatens, New York City
Oliver Antrum Hall, Juneau, Alaska
John Milton Hansen, Jamestown, N. Dak.
Alfred Reid Henderson, Jersey City, N. J.
Matthew Charles Hendrie, Denver, Colo.
Hugh Browning Holmes, New York City
Clarence Scott Howell, New York City
Thomas Paul Humphrey, Kansas City, Mo.

James O'Connor Hunt, Knoxville, Tenn.
Joseph Greenwood Hunter, Washington, D. C.
Nejib Hovhanness Jebejian, Youngstown, Ohio
John Monroe Johnson, Am. Exp. Force, France
William George Juengst, East St. Louis, Ill.
Swan August Kalberg, Springfield, Mass.
Edward Mathias Killough, Bethlehem, Pa.
Harry Anderson La Rue, Kansas City, Mo.
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Reynold Ferdinand Melin, Fort Monroe, Va.
Robert Patchen Miller, Los Angeles, Cal.
Henry Lewis Moeller, Hoboken, N. J.
Gilbert Morrison, State College, Pa.
Charles Augustine Mullen, Montreal, Que., Canada
Egbert Stephen Needham, Livingston, Mont.
Claire Green Norris, Coney, Kans.
Robert Norris, Ann Arbor, Mich.
Owen Hugh O'Neill, Santa Barbara, Cal.
George Carrol Oxer, Arrochar, N. Y.
Seth Perkins, Jr., St. Augustine, Fla.
Charles Aloysius Petry, Urbana, Ill.
Floyd Jacob Pitcher, New Haven, Conn.
Ben Tarver Quarles, Blooming Grove, Tex.
John Revell, New York City.
John Turner Rice, Imperial, Cal.
Leo Bond Roberts, Washington, D. C.
Glenn A. Ruggles, Peoria, Ill.
George Christian Schoenberger, Natchez, Miss.
Henry Darcy Scudder, Jr., Newark, N. J.
David Sutton, Watertown, Mass.
Ernest Osgood Sweetser, St. Louis, Mo.
Alfred Tamm, Mission, Tex.
Robert Neel Van Winkle, Toledo, Ohio
Jeptha A. Wade, Ann Arbor, Mich.
Edward Whitwell, New York City
Louis Arthur Wilson, Sibley, Iowa
William Charles Wolf, Belleville, Ill.

As Associate

William Thomas Headley, Philadelphia, Pa.

As Juniors

Harry Conrad Francis Alber, Green Island, N. Y.
Harold Crusius Bird, Chester, Pa.
Stephen Greville de Mahy, St. George, Grenada
Malcolm Duncan, Reading, Pa.
George Edmund Gambier-Bousfield, New York City
Reinhold Bernhard Hansen, San Francisco, Cal.
William Trenholm Hopkins, Savannah, Ga.
Edwin August Humann, Oakland, Cal.
Samuel Pérry Laverty, Lindsay, Cal.
Joseph Irwin Levy, Bayonne, N. J.
Vernon Lemley Logan, Hutchinson, Kans.
Charles Harold McCrea, Fort Sheridan, Ill.
William John McIntyre, Morristown, N. J.
Charles Philip Mills, Philadelphia, Pa.
Arthur Franklin Perry, Jr., Jacksonville, Fla.
Harold Ellwood Pride, Chicago, Ill.
Raymond Castle Reese, Pittsfield, Mass.
William Hepbourne Russell, Fort Benjamin Harrison, Ind.
Donald Abram Smith, Topinabee, Mich.
Ralph Robert Smith, Batavia, N. Y.
Jean-Baptiste Octave Sneeden, Glasgow, Scotland

The Secretary announced the transfer of the following candidates on November 27th, 1917:

From Associate Member to Member

James Ray Aikenhead, New York City
Guy Mannering Bassel, Maryville, Tenn.
Clinton Llewellyn Cole, Meriden, Conn.
Erle Long Cope, San Francisco, Cal.
Julian Clarence Feild, Denison, Tex.
Thomas Fleming, Jr., Pittsburgh, Pa.
William Goldsmith, New Hampton, N. Y.
William Henry Hall, New Britain, Conn.
Albert Harrison Hinkle, Columbus, Ohio.
David Clayton Johnson, New York City
Royal John Mansfield, New York City
Herman Stabler, Washington, D. C.
Jesse Clark Wright, Los Angeles, Cal.

From Junior to Associate Member

William Selby Allan, Petersburg, Ind.
Dumont Beerbower, Cleveland, Ohio

ALBERT EDWARD BROKER, Chicago, Ill.
GEORGE BRYAN, JR., Duluth, Minn.
WILLIAM FISHER DRURY, Waterville, Me.
LESTER GURNEY, Springfield, Mass.
GEORGE ALFRED HELMSTETTER, Syracuse, N. Y.
ALFRED SPARKS HIRZEL, Wilmington, Del.
SAMUEL FRANK NEWKIRK, Bayside, N. Y.
HARRY BROWNLEY PEARSON, JR., Swedeland, Pa.
GILBERT COBB STAEHLE, Minneapolis, Minn.
JOHN WRIGHT TAUSSIG, New York City
EARLE CHESTER WAITE, South Bethlehem, Pa.
WILLIAM ALBERT YEO, Pearl Lagoon, Nicaragua

The Secretary announced the following deaths:

JOHN GRIFFITHS BROWN, of Philadelphia, Pa., elected Associate, July 9th, 1906; Member, September 3d, 1913; died October 22d, 1917.

HARVEY COOPER MILLER, of Brooklyn, N. Y., elected Member, June 5th, 1901; died November 23d, 1917.

NELSON JAMES WELTON, of Waterbury, Conn., elected Fellow, January 20th, 1873; died June 5th, 1917.

Adjourned.

December 19th, 1917.—Because of the necessity of going to press with this number of *Proceedings* in advance of this meeting, the publication of its minutes must be deferred until January, 1918.

OF THE BOARD OF DIRECTION

(Abstract)

November 27th, 1917.—The Board met at 10.30 P. M., immediately after the adjournment of the Membership Committee; President Pegram in the chair; Chas. Warren Hunt, Secretary; and present, also, Messrs. Flinn, Humphreys, Kittredge, Noble, and Tillson.

Ballots for membership were canvassed, resulting in the election of 10 Members, 79 Associate Members, 1 Associate, and 21 Juniors, and the transfer of 14 Juniors to the grade of Associate Member.

Thirteen Associate Members were transferred to the grade of Member.

A Report from the Membership Committee was received and acted upon.

Adjourned.

SOCIETY ITEMS OF INTEREST

Resolutions Adopted by the Executive Committee American Society of Civil Engineers, November 27th, 1917.

Whereas, It has come to be generally recognized that the prosecution of the present war to a successful issue requires that all forces, especially human forces, must be co-ordinated and classified, and that technically trained men must be employed on lines along which their particular training may be used to the greatest advantage; and

Whereas, The messages that have come to the Engineers of the United States, through the several missions from our Allies, have confirmed and reinforced the belief that the service of the Engineer is essential and must be properly applied; and

Whereas, The supply of technically trained Engineers for present and future usefulness must be continuous; be it

Resolved, That, in the opinion of the Board of Direction of the American Society of Civil Engineers, all possible consideration should be given to students, who are certified by Engineering Schools of recognized reputation as having evidenced their capacity to become Engineers, to continue their education to graduation, if possible, and, in any event, until necessity demands their call to active service.

These resolutions were forwarded to the Secretary of War, and to other officials, and resolutions covering the same ground have been presented from Engineering Council, from the Society for the Promotion of Engineering Education, and doubtless from other bodies. The result has been that the following has been added by the Provost Marshal General to Section 151 of the Selective Service Regulations:

"Under such regulations as the Chief of Engineers may prescribe a proportion of the students pursuing an engineering course in one of the approved technical engineering schools listed in the War Department, as named by the school faculty, may enlist in the Enlisted Reserve Corps of the Engineer Department and thereafter upon presentation by the registrant to his local board of a certificate of enlistment, such certificate shall be filed with the Questionaire and the registrant shall be placed in Class V on the ground that he is in the military service of the United States."

The regulations of the Chief of Engineers limit this privilege to those students to whom the school issues the following certificate properly attested by the President of the school:

I hereby certify that ....
is a regular student of the .... class in good standing,
as a candidate for an Engineering degree at ....
.... and that in the judgment of the faculty of this

school, based upon his academic record, supplemented by his relations with fellow students and by observation of his instructors, he may fairly be regarded as deserving a place in the first third qualitatively of the young men graduating from this institution during the past ten years.

The above was approved by the Secretary of War, December 8th, 1917.

The following letter has been received:

"WAR DEPARTMENT
"OFFICE OF THE CHIEF OF ENGINEERS
"WASHINGTON

"DECEMBER 6, 1917.

"DR. CHARLES W. HUNT,
"Secretary, AMERICAN SOCIETY OF CIVIL ENGINEERS,
"33 West Thirty-ninth Street,
"New York City.

"MY DEAR DR. HUNT:

"1. Receipt is acknowledged of your letter of December 1, 1917, addressed to the Secretary of War and transmitting copy of the resolution adopted by the Executive Committee of your Society at its meeting of November 27, 1917. Secretary Baker has directed me to reply to this communication.

"2. The question of the exemption of engineering students has been under advisement by this office for some time. Considerable correspondence has been received from the heads of different engineering colleges and others interested in the advancement of engineering education.

"3. Several weeks ago a memorandum was submitted by this office to the Provost Marshal General recommending that a certain proportion of the engineering students of all engineering schools of the country accredited by the War Department be exempted. The details are being worked out by the Provost Marshal General's office and it is hoped that suitable regulations may be framed to afford relief before the calling of the next draft.

"4. The War Department has been much gratified by the co-operation which it has received from the Engineering Societies and is anxious to do all it can, consistent with the exigencies of war, to reduce interference with educational activities, important industries and other activities of the country at large to a minimum.

"Very sincerely,

"W. M. BLACK,
"Major General, Chief of Engineers."

ANNOUNCEMENTS

The Reading Room of the Society is open from 9 A. M. to 10 P. M., every day, except Sundays, Fourth of July, Thanksgiving Day, and Christmas Day.

FUTURE MEETINGS

January 2d, 1918.—8.30 P. M.—A regular business meeting will be held, and a paper by Benjamin F. Groat, M. Am. Soc. C. E., entitled "Ice Diversion, Hydraulic Models, and Hydraulic Similarity", will be presented for discussion.

This paper is printed in this number of *Proceedings*.

February 2d, 1918.—8.30 P. M.—This will be a regular business meeting. A paper by F. W. Gardiner and S. Johannesson, Members, Am. Soc. C. E., entitled, "Manhattan Elevated Railway Improvements", will be presented for discussion.

This paper is printed in this number of *Proceedings*.

ANNUAL MEETING

The Sixty-fifth Annual Meeting will be held at the Society House, on Wednesday and Thursday, January 16th and 17th, 1918.

COMMITTEE OF ARRANGEMENTS

ARTHUR S. TUTTLE,

BEVERLY R. VALUE, JAMES F. SANBORN,

CLIFFORD M. HOLLAND, CHAS. WARREN HUNT.

The Business Meeting will be called to order at 10 o'clock on Wednesday morning. The Annual Reports will be presented, Officers for the ensuing year elected, members of the Nominating Committee appointed, Reports of Special Committees presented for discussion, Proposed Amendments of the Constitution considered, and other business transacted.

REPORTS OF SPECIAL COMMITTEES

The Final Reports of the Special Committees on Materials for Road Construction and on Steel Columns and Struts, and a Progress Report of the Special Committee on the Regulation of Water Rights, are printed in this number of *Proceedings* with the Papers and Discussions.

SEARCHES IN THE LIBRARY

In January, 1902, the Secretary was authorized to make searches in the Library, upon request, and to charge therefor the actual cost to the Society for the extra work required. Since that time many searches have been made, and bibliographies and other information on special subjects furnished.

The resulting satisfaction, to the members who have made use of the resources of the Society in this manner, has been expressed frequently, and leaves little doubt that if it were generally known to the membership that such work would be undertaken, many would avail themselves of it.

The cost is trifling compared with the value of the time of an engineer who looks up such matters himself, and the work can be performed quite as well, and much more quickly, by persons familiar with the Library.

In asking that such work be undertaken, members should specify clearly the subject to be covered, and whether references to general books only are desired, or whether a complete bibliography, involving search through periodical literature, is desired.

It sometimes happens that references are found which are not readily accessible to the person for whom the search is made. In that case the material may be reproduced by photography, and this can be done for members at the cost of the work to the Society, which is small. This method is particularly useful when there are drawings or figures in the text which would be very expensive to reproduce by hand.

A list of 989 bibliographies made in the Library, giving the cost of each, was published in Vol. LXXX of *Transactions*.

Since October 1st, 1916, the Library of the American Society of Civil Engineers has ceased to exist, as such, having been merged with the Libraries of the Mining, Mechanical, and Electrical Engineers, and become a part of the Library of the United Engineering Society. There were 67 000 accessions, which were not duplicates, turned over to that Library.

Hereafter, therefore, requests for research should be addressed to the Librarian, Engineering Societies Library, 29 West 39th Street, New York City.

PAPERS AND DISCUSSIONS

Members and others who take part in the oral discussions of the papers presented are urged to revise their remarks promptly. Written communications from those who cannot attend the meetings should be sent in at the earliest possible date after the issue of a paper in *Proceedings*.

All papers accepted by the Publication Committee are classified by the Committee with respect to their availability for discussion at meetings.

Papers which, from their general nature, appear to be of a character suitable for oral discussion will be published as heretofore in *Proceedings*, and set down for presentation to a future meeting of the Society, and on these oral discussions, as well as written communications, will be solicited.

All papers which do not come under this heading, that is to say, those which from their mathematical or technical nature, in the opinion of the Committee, are not adapted to oral discussion, will not be scheduled for presentation to any meeting. Such papers will be published in *Proceedings* in the same manner as those which are to be presented at meetings, but written discussions only will be requested for subsequent publication in *Proceedings* and with the paper in the volumes of *Transactions*.

The Board of Direction has adopted rules for the preparation and presentation of papers, which will be found on page 429 of the August, 1913, *Proceedings*.

LOCAL ASSOCIATIONS OF MEMBERS OF THE AMERICAN SOCIETY OF CIVIL ENGINEERS

San Francisco Association, Organized 1905.

J. D. Galloway, President; E. T. Thurston, Secretary-Treasurer, 57 Post Street, San Francisco, Cal.

The San Francisco Association of Members of the American Society of Civil Engineers holds regular bi-monthly meetings, with banquet, and weekly informal luncheons. The former are held at 6 P. M., at the Palace Hotel, on the third Tuesday of February, April, June, August, October, and December, the last being the Annual Meeting of the Association.

Informal luncheons are held at 12.30 P. M., every Wednesday, and the place of meeting may be ascertained by communicating with the Secretary.

The by-laws of the Association provide for the extension of hospitality to any member of the Society who may be temporarily in San Francisco, and any such member will be gladly welcomed as a guest.

Colorado Association, Organized 1908.

Robert Follansbee, President; L. R. Hinman, Secretary-Treasurer, 1400 West Colfax Avenue, Denver, Colo.

The meetings of the Colorado Association of Members of the American Society of Civil Engineers (Denver, Colo.) are held on the second Saturday of each month, except July and August. The hour and place of meeting are not fixed, but this information will be furnished on application to the Secretary. The meetings are usually preceded by an informal dinner. Members of the American Society of Civil Engineers will be welcomed at these meetings.

Weekly luncheons are held on Wednesdays at 12.30 P. M., at Daniel's and Fisher's.

Visiting members are urged to attend the meetings and luncheons.

Atlanta Association, Organized 1912.

B. M. Hall, President; W. C. Spiker, Secretary-Treasurer, 1408 Candler Building, Atlanta, Ga.

Baltimore Association, Organized 1914.

Mason D. Pratt, President; Charles J. Tilden, Secretary-Treasurer, The Johns Hopkins University, Baltimore, Md.

Cleveland Association, Organized 1914.

W. J. Watson, President; George H. Tinker, Secretary-Treasurer, 516 Columbia Building, Cleveland, Ohio.

Detroit Association, Organized 1916.

T. A. Leisen, President; Clarence W. Hubbell, Secretary, 2348 Penobscot Building, Detroit, Mich.

The regular meetings of the Association are held on the second Friday of December, April, and October, the last being the Annual Meeting.

District of Columbia Association, Organized 1916.

A. P. Davis, President; John C. Hoyt, Secretary-Treasurer, U. S. Geological Survey, Washington, D. C.

Duluth Association, Organized 1917.

F. E. House, President; Walter G. Zimmermann, Secretary, Wolvin Building, Duluth, Minn.

The regular meetings of the Association are held at noon on the third Monday of each month (usually at the Kitchi Gamma Club), with luncheon, followed by a short business session and reading of papers. Visiting members of the American Society of Civil Engineers can secure from the Secretary definite information relating to the meetings, at which they will be welcomed. The Annual Meeting is held on the third Monday in May.

(Abstract of Minutes of Meeting)

November 19th, 1917.—The meeting was called to order; Vice-President J. H. Darling in the chair; H. C. Ash, acting as Secretary; and present, also, 22 members and 1 guest.

The Auditing Committee reported that the books of the Association were in order, and had been re-opened and turned over to Treasurer Carson. The Committee also suggested that the Association remit the dues of all members absent on active military service and that their names be carried on the rolls as long as they claimed Duluth as their home.

The report was adopted and the Committee discharged.

The resolutions adopted by the Board of Direction relative to the 8% super-tax on incomes of professional men, under the new revenue law, were read, and, on motion, duly seconded, it was decided to endorse the resolution and protest the tax as unjust, the protest to be signed by the President and Secretary and sent to representatives of the State of Minnesota in the Senate and House of Representatives and that, in addition, all members of the Association be urged to make personal remonstrances.

A committee was appointed to draw up and present a resolution of sympathy and condolence to Mr. Zimmermann on the death of his wife.

On motion, duly seconded, it was decided that the President and Secretary constitute a Committee to see that flowers or other appropriate remembrances be sent in the event of sickness or death in the family of any member.

Mr. H. L. Christie addressed the meeting extemporaneously on the reconstruction of bridges on the Central Railroad of Peru, with which work he was connected in 1909 and 1910. On motion, duly seconded, Mr. Christie was requested to put his talk in the form of a paper for the Society records.

Mr. Coleman also addressed the meeting relative to the work done by the Corps of Engineers, U. S. A., since the United States entered the war.

Adjourned.

Illinois Association, Organized 1916.

C. F. Loweth, President, Chicago, Ill.

The regular meetings of the Association are held on the second Monday of March, June, September, and December, the last being the Annual Meeting. The hour and place of meeting are not fixed, but this information will be furnished on application to the President.

Louisiana Association, Organized 1914.

W. B. Gregory, President; Charles W. Okey, Secretary, Tulane University, New Orleans, La.

The regular meetings of the Association are held at The Cabildo, New Orleans, La., on the first Monday of January, April, July, and October.

Nebraska Association, Organized 1917.

Frank T. Darrow, President; Homer V. Knouse, Secretary-Treasurer, 115 City Hall, Omaha, Nebr.

Regular meetings of the Association are held on the first Saturday of each month, except July and August, and at such places as may be appointed from time to time by the Executive Committee. The Annual Meeting is held in Lincoln, Nebr., on the second Friday in January.

An "Engineers' Round Table" is reserved daily for luncheon, at Courtenay's Restaurant, 17th and Douglas Streets, Omaha, to which all engineers are invited. Visiting members of the Society are especially urged to communicate with the Secretary when in the city.

(Abstract of Minutes of Meeting)

November 10th, 1917.—The meeting was called to order at 8.30 P. M., at the Lincoln Hotel, Lincoln, Nebr.; Vice-President Darrow in the chair; Homer V. Knouse, Secretary; and present also, 8 members and 1 guest.

The minutes of the September and October meetings and of the special meeting of July 28th, 1917, were read and approved.

The Secretary presented a letter from Chas. Warren Hunt, Secretary of the Society, pertaining to a Resolution adopted by the Board of Direction on November 1st, 1917, in regard to the War Revenue Act of 1917.

A letter from Henry R. Buck, M. Am. Soc. C. E., relative to the proposed establishment of Student Branches of the Society, was also read.

On motion, duly seconded, the Secretary was instructed to call the attention of the members of the Association to the matter referred to in Mr. Hunt's letter, and it was also decided that the Resolution be discussed and acted on at the December meeting.

On motion, duly seconded, the Secretary was instructed to advise Mr. Buck as to the action of the Association at its meeting of March 3d, 1917, in regard to Student Branches.

On motion, duly seconded, the Secretary was instructed to communicate to Capt. Scott King and Capt. H. E. McClintock, the congratulations of the Association on their appointment to the Officers Reserve Corps of the United States Army, and to remit their dues during their period of service with the Government.

The character of the programme to be provided at future meetings of the Association was discussed by Messrs. Campen, Standeven, Grant, Dobson, Hershey, Walsh, and Mickey. No formal action was taken, but it was the opinion of those present that a definite subject should be assigned for discussion, with short papers by one or two members, and that frequent trips to works under construction or being operated, with short descriptive papers, would be a desirable feature.

On motion, duly seconded, it was decided that the December meeting of the Association be held in Omaha, that the programme should pertain to Methods of Water Purification, and that there should be an inspection trip to the Alum Cake Manufacturing Plant, at the Florence Station of the Metropolitan Water District of Omaha. Messrs. Prince and Knouse were appointed a Committee to take charge of the meeting.

Mr. George A. Campen presented an interesting description of the organization and work accomplished at the Fort Omaha Balloon School by Mr. E. Wickham, the Contractor for this cantonment.

Adjourned.

Northwestern Association, Organized 1914.

George L. Wilson, President; Ralph D. Thomas, Secretary, 508 South First Street, Minneapolis, Minn.

Philadelphia Association, Organized 1913.

Henry H. Quimby, President; C. W. Thorn, Secretary, 1313 South Broad Street, Philadelphia, Pa.

The regular meetings of the Association are held at the Engineers' Club of Philadelphia, 1317 Spruce Street, on the First Monday in January, April, and October, the last being the Annual Meeting.

(Abstract of Minutes of Meeting)

October 1st, 1917.—The Annual Meeting was called to order at the Engineers' Club, at 8.30 P. M.; President Samuel T. Wagner in the chair; C. W. Thorn, Secretary; and present, also, 76 members and guests.

Messrs. George S. Webster and Richard L. Humphrey were appointed Tellers to canvass the ballots for officers.

A resolution relative to the establishment of Student Branches of the Society was unanimously approved as adopted by the Board of Direction of the Association.

George H. Pegram, President of the Society, addressed the meeting on "The Construction of the New York Subways", illustrating his remarks with lantern slides. At the conclusion of his address, Mr. Pegram received a rising vote of thanks for his paper.

The Tellers appointed to canvass the ballots for officers reported the following elections: President, Henry H. Quimby; Vice-President, John Meigs; Directors, W. L. Stevenson and John F. Murray; and Treasurer, S. M. Swaab.

Mr. Quimby then took the chair and accepted the duties of his office.

Adjourned.

Portland, Ore., Association, Organized 1913.

J. P. Newell, President; J. A. Currey, Secretary, 194 North 13th Street, Portland, Ore.

St. Louis Association, Organized 1888 (1914).

J. A. Ockerson, President; C. M. Daily, Secretary-Treasurer, 34 East Grand Avenue, St. Louis, Mo.

The Annual Meeting of the Association is held on the fourth Monday in November in the Auditorium of the Engineers Club of St. Louis. The time and place of other meetings are not fixed, but this information will be furnished on application to the Secretary.

(Abstract of Minutes of Meeting)

November 26th, 1917.—The Annual Meeting was called to order at the American Hotel Annex; President Ockerson in the chair; C. M. Daily, Secretary.

The report of the Secretary-Treasurer was read and approved.

A letter from Chas. Warren Hunt, Secretary of the Society, relative to the Resolution of the Executive Committee of the Board of Direction on the War Revenue Act of 1917, together with the Resolution, was read, and the Act was discussed.

On motion duly seconded, the Secretary was instructed to write to the Board of Direction, advising it that the Association would do all in its power to have the Act amended.

The report of the Nominating Committee was read, and, on motion, duly seconded, the nominations were closed.

A temporary Chairman was appointed to conduct the election, and, on motion, duly seconded, the Secretary was instructed to cast the ballot.

The following officers were elected: President, John A. Ockerson; First Vice-President, Edward E. Wall; Second Vice-President, W. S. Mitchell; and Secretary-Treasurer, C. M. Daily.

Capt. A. A. Poland, Quartermaster Corps, U. S. Reserves, addressed the meeting relative to the efforts being made by the various municipalities and the Government to revive river traffic.

Mr. H. J. Pfeifer described the effect of the war on railroad operation and the co-operation of the railroads with the various departments of the Government.

Adjourned.

San Diego Association, Organized 1915.

W. J. Gough, President; J. R. Comly, Secretary-Treasurer, 4105 Falcon Street, San Diego, Cal.

Seattle Association, Organized 1913.

Joseph Jacobs, President; Carl H. Reeves, Secretary-Treasurer, 444 Henry Building, Seattle, Wash.

The regular meetings of the Association are held at 12.15 P. M., on the last Monday of each month, at The Frye Hotel.

Southern California Association, Organized 1914.

H. Hawgood, President; H. W. Dennis, Secretary, 329 San Fernando Building, Los Angeles, Cal.

The Southern California Association of Members of the American Society of Civil Engineers (Los Angeles, Cal.) holds regular bi-monthly meetings with banquet, at Hotel Clark, on the second Wednesday of February, April, June, August, October, and December, the last being the Annual Meeting of the Association.

Informal luncheons are held at 12.15 P. M. every Wednesday, and the place of meeting may be ascertained from the Secretary.

The by-laws of the Association provide for the extension of hospitality to any member of the Society who may be temporarily in Los Angeles, and any such member will be gladly welcomed as a guest at any of the meetings or luncheons.

Spokane Association, Organized 1914.

J. C. Ralston, President; B. J. Garnett, Secretary, City Hall, Spokane, Wash.

The regular meetings of the Association are held on the second Friday of each month, except July and August. The hour and place of meeting are not fixed, but this information will be furnished on application to the Secretary.

Visiting members are invited to attend the meetings and luncheons.

Texas Association, Organized 1913.

John B. Hawley, President; J. F. Witt, Secretary, Dallas, Tex.

Utah Association, Organized 1916.

George L. Swendsen, President; H. S. Kleinschmidt, Secretary-Treasurer, 306 Dooly Building, Salt Lake City, Utah.

The Annual Meeting of the Association is held on the first Wednesday in April. The time of other meetings is not fixed, but this information will be furnished on application to the Secretary.

PRIVILEGES OF ENGINEERING SOCIETIES EXTENDED TO MEMBERS OF THE AMERICAN SOCIETY OF CIVIL ENGINEERS

Members of the American Society of Civil Engineers will be welcomed by the following Engineering Societies, both to the use of their Reading Rooms, and at all meetings:

American Institute of Electrical Engineers, 25 West Thirty-ninth Street, New York City.

American Institute of Mining Engineers, 25 West Thirty-ninth Street, New York City.

American Society of Mechanical Engineers, 25 West Thirty-ninth Street, New York City.

Assogiacão dos Engenheiros Civis Portuguezes, Lisbon, Portugal.

Australasian Institute of Mining Engineers, Melbourne, Victoria, Australia.

Boston Society of Civil Engineers, 715 Tremont Temple, Boston, Mass.

Brooklyn Engineers' Club, 117 Remsen Street, Brooklyn, N. Y.

Canadian Society of Civil Engineers, 176 Mansfield Street, Montreal, Que., Canada.

Civil Engineers' Society of St. Paul, Public Library, St. Paul, Minn.

Cleveland Engineering Society, Chamber of Commerce Building, Cleveland, Ohio.

Cleveland Institute of Engineers, Middlesbrough, England.

Dansk Ingeniorforening, Amaliegade 38, Copenhagen, Denmark.

Detroit Engineering Society, 46 Grand River Avenue, West, Detroit, Mich.

Engineering Association of Nashville, Commercial Club Building, Nashville, Tenn.

Engineering Societies Club of Hawaii, E. F. Cykler, Secretary, Honolulu, Hawaii.

Engineers and Architects Club of Louisville, 1412 Starks Building, Louisville, Ky.

Engineers' Club of Baltimore, 6 West Eager Street, Baltimore, Md.

Engineers' Club of Kansas City, E. B. Murray, Secretary, 920 Walnut Street, Kansas City, Mo.

Engineers' Club of Minneapolis, 17 South Sixth Street, Minneapolis, Minn.

Engineers' Club of Philadelphia, 1317 Spruce Street, Philadelphia, Pa.

Engineers' Club of St. Louis, 3817 Olive Street, St. Louis, Mo.

Engineers' Club of Toronto, 96 King Street, West, Toronto, Ont., Canada.

Engineers' Club of Trenton, Trent Theatre Building, 12 North Warren Street, Trenton, N. J.

Engineers' Society of Northeastern Pennsylvania, 415 Washington Avenue, Scranton, Pa.

Engineers' Society of Pennsylvania, 31 South Front Street, Harrisburg, Pa.

Engineers' Society of Western Pennsylvania, 568 Union Arcade Building, Pittsburgh, Pa.

Florida Engineering Society, J. R. Benton, Secretary, Gainesville, Fla.

Institute of Marine Engineers, The Minories, Tower Hill, London, E., England.

Institution of Civil Engineers, Great George Street, Westminster, S. W., London, England.

Institution of Engineers of the River Plate, Calle 25 de Mayo 195, Buenos Aires, Argentine Republic.

Institution of Naval Architects, 5 Adelphi Terrace, London, W. C., England.

Junior Institution of Engineers, 39 Victoria Street, Westminster, S. W., London, England.

Koninklijk Instituut van Ingenieurs, The Hague, The Netherlands.

Louisiana Engineering Society, State Museum Building, Chartres and St. Ann Streets, New Orleans, La.

Memphis Engineers' Club, Memphis, Tenn.

Midland Institute of Mining, Civil and Mechanical Engineers, Sheffield, England.

Montana Society of Engineers, Butte, Mont.

North of England Institute of Mining and Mechanical Engineers, Newcastle-upon-Tyne, England.

Oregon Society of Civil Engineers, Portland, Ore.

Pacific Northwest Society of Engineers, 803 Central Building, Seattle, Wash.

Rochester Engineering Society, Rochester, N. Y.

Sociedad Colombiana de Ingenieros, Bogota, Colombia.

Sociedad de Ingenieros del Peru, Lima, Peru.

Societe des Ingenieurs Civils de France, 19 rue Blanche, Paris, France.

Society of Engineers, 17 Victoria Street, Westminster, S. W., London, England.

Southwestern Society of Engineers, C. E. Barglebaugh, Secretary, 703 First National Bank Building, El Paso, Tex.

Svenska Teknologforeningen, Brunkebergstorg 18, Stockholm, Sweden.

Tekniske Forening, Vestre Boulevard 18–1, Copenhagen, Denmark.

Vermont Society of Engineers, George A. Reed, Secretary, Montpelier, Vt.

Western Society of Engineers, 1735 Monadnock Block, Chicago, Ill.

ACCESSIONS TO THE UNITED ENGINEERING SOCIETY LIBRARY

(From November 2d to December 1st, 1917)

DONATIONS*

The statements made in these notices are taken directly from the books themselves, and this Society is not responsible for them.

PRACTICAL ELECTRICITY.

By Terrell Croft. N. Y., McGraw-Hill Book Co., Inc.; Lond., Hill Publishing Co., Ltd., 1917. 14 + 646 pp., 548 illus., 8 x 6 in., cloth. $2.50.

Written primarily to present the fundamental facts and theories of electricity and its applications for the use of students who desire a working knowledge of the subject, and secondarily for those who wish to review and reconstruct their concepts of electric and magnetic phenomena in accordance with modern theory and practice.

NAVIGATION.

By Harold Jacoby. N. Y., The Macmillan Co., 1917. 11 + 330 pp., 22 illus., 8 x 5 in., cloth. $2.25.

Intended to present the methods approved by the most reliable modern authorities in a manner which can be understood by those without formal mathematical and astronomical knowledge, and yet with sufficient completeness to make possible the navigation of a ship in any ocean not very near the north and south poles, without other books or tabular works, excepting only the Nautical Almanac for the year in which the voyage is made.

MACHINERY'S ENCYCLOPEDIA:

A Work of Reference Covering Practical Mathematics and Mechanics, Machine Design, Machine Construction and Operation, Electrical, Gas, Hydraulic, and Steam Power Machinery, Metallurgy, and Kindred Subjects in the Engineering Field. Compiled and Edited by Erik Oberg and Franklin D. Jones. N. Y., The Industrial Press; Lond., The Machinery Publishing Co., Ltd., 1917. 7 vol., illus., 11 x 8 in., ½ leather. $36.00.

A convenient reference library for mechanical engineers, shop foremen, draftsmen, machinists, etc., giving a general survey of the mechanical field. Especial prominence is given to practice, and mathematical discussions are abbreviated when possible. The material is based on the articles published in *Machinery*, but these have been supplemented by compilations from standard sources and by numerous signed articles. The work contains more than 3 000 line illustrations, and is arranged alphabetically, with frequent cross-references, and is also provided with an index to subjects not brought out in the main headings. A guide to systematic reading is appended to the work.

ACQUIRING WINGS:

A Text on the Basic Principles Governing the Design and Operation of Modern Air Craft. By William B. Stout. N. Y., Moffat, Yard & Co., 1917. 57 pp., 11 illus., 7 x 5 in., cloth. 75 cents.

The original draft of this volume was written for a small corps of men from various motor-car plants, who were sent abroad by the Government at a certain stage in the war, to study aircraft production and its problems. Explains briefly and non-mathematically the principles of the airplane, its design, construction, and operation.

* Unless otherwise specified, books in this list have been donated by the publisher.

HOW TO FLY

(The Flyer's Manual): A Practical Course of Training in Aviation. By Capt. D. Gordon E. Re Vley. Arranged by Glad Lewis. San Francisco, Paul Elder and Co., 1917. 100 pp., 1 por., 6 x 4 in., cloth. $1.00.

Presents a series of graduated exercises for training aviators, intended to give the student confidence and self-reliance.

GLOSSARY OF AVIATION TERMS (TERMES D'AVIATION).

Compiled by Victor W. Page and Paul Montariol. N. Y., The Norman W. Henley Publishing Co., 1917. 94 pp., 14 pl., 8 x 5 in., boards. $1.00.

Divided into an English-French and a French-English section. Includes the terms in general use in France and America. The plates show the details of the airplane and its equipment, each part bearing its name in both languages.

AN ELEMENTARY OUTLINE OF MECHANICAL PROCESSES;

Giving a Brief Account of the Materials Used in Engineering Construction and of the Essential Features in the Methods of Producing Them, also Describing Shop Processes and Equipment for the Shaping of Metals into Forms for Engineering and General Uses. Arranged for the Instruction of Midshipmen at the U. S. Naval Academy and for Students in General. By G. W. Danforth. Annapolis, The United States Naval Institute, 1917. 423 pp., 270 illus., 9 x 6 in., cloth.

Intended to show completely, yet briefly, the steps of metal manufacture from the ore to the finished product.

THE TECHNICAL ANALYSIS OF BRASS

And the Non-Ferrous Alloys. By William B. Price and Richard K. Meade. 2d ed., rev. and enl. N. Y., John Wiley and Sons, Inc.; Lond., Chapman & Hall, Ltd., 1917. 9 + 376 pp., 25 illus., 8 x 5 in., cloth. $3.00.

A collection of accurate, quick methods intended to contain a complete treatment of the subject in one volume and to occupy a place in the analytical laboratory corresponding to that occupied by Blair's "Analysis of Iron." Contents: Introduction; Determination of the Metals; Some Applied Examples of Alloy Analysis; Control and Analysis of Plating Solutions.

HAND GRENADES:

A Handbook on Rifle and Hand Grenades; Compiled and Illustrated by Major Graham M. Ainslie. N. Y., John Wiley & Sons, Inc.; Lond., Chapman & Hall, Ltd., 1917. 59 pp., 23 illus., 7 x 5 in., cloth. $1.25.

Each type is briefly described, and concise directions are given for firing it. Sectional drawings show the construction. The explanations are definite and concise, and intended for persons with little knowledge of grenades.

THE BOOK OF THE MACHINE GUN.

By Maj. F. V. Longstaff and A. Hilliard Atteridge. Lond., Hugh Rees, Ltd., 1917. 14 + 337 + 84 pp., 69 illus., 16 pl., 9 x 6 in., cloth. $3.50. (Gift of Dodd, Mead & Co.)

A history and description of the evolution of the machine gun in its various types and of the evolution of machine-gun tactics. Includes a bibliography and a list of British patents. The illustrations are arranged in chronological order and provide a graphic presentation of the progress of machine-gun design and construction.

FIELD FORTIFICATION:

A Study of the Western Front in Europe, 1914-1916. Reprinted from the *Infantry Journal,* 1917. Wash., The United States Infantry Association, 1917. 106 pp., 39 illus., 10 x 7 in., cloth. $1.00.

A description of the general principles and practice of to-day, based on authentic information. Intended for use in educating soldiers of the United States.

THE ELEMENTS OF COAL MINING.

By Daniel Burns. Lond., Edward Arnold, 1917. 7 + 236 pp., 113 illus., 7 x 5 in., cloth. $1.10. (Gift of Longmans, Green & Company.)

An introductory course intended specifically for schoolboys who intend to become coal miners. Gives a general idea of how coal occurs, how it is mined, how the miner is safeguarded while at work, and how coal is distributed and used.

ORE MINING METHODS:

Comprising Descriptions of Methods of Support in Extraction of Ore, Detailed Descriptions of Methods of Development of Mines, of Stoping and Mining in Narrow and Wide Veins and Bedded and Massive Deposits, Including Stull and Square-Set Mining, Filling and Caving Methods, Open Cut Work and a Discussion of Costs of Mining. By Walter R. Crane. 2d ed. N. Y., John Wiley & Sons, Inc.; Lond., Chapman & Hall, Ltd., 1917. 13 + 277 pp., 84 illus., 9 x 6 in., cloth. $3.50.

A systematic, detailed description of methods of mining ore, illustrated by photographs of mine models and accompanied by statements of the applications of each method, its advantages and disadvantages. Bibliographies are included with the chapters. This edition has been revised and enlarged, and a chapter on the development of mines has been added.

PRACTICAL BANKING.

By O. Howard Wolfe. Chic., La Salle Extension University, 1917. 11 + 290 pp., 95 illus., 8 x 6 in., leather. $2.00.

A textbook for students, covering the fundamental principles, but omitting those subjects which can be mastered only by experience.

APPLIED METHODS OF SCIENTIFIC MANAGEMENT.

By Frederic A. Parkhurst. 2d ed. N. Y., John Wiley & Sons, Inc.; Lond., Chapman & Hall, Ltd., 1917. 12 + 337 pp., 55 illus., 9 pl., 9 x 6 in., cloth. $2.00.

A detailed description of the application of methods, illustrated by the history of their use in the factory of the Ferracute Machine Company. The second edition is a reprint of the first, with twelve additional pages containing letters from the company mentioned, which testify to the success of the plan.

INORGANIC CHEMISTRY.

By Horace G. Byers. N. Y., Chic. & Bost., Charles Scribners Sons (copyright 1917). 651 pp., 138 illus., 8 x 5 in., cloth. $2.25.

A textbook for beginners of college grade, intended for use in classes which include those studying the subject as a complement to a liberal education and those undertaking it as a tool to be used in various professions.

A LABORATORY MANUAL OF GENERAL CHEMISTRY.

By Horace G. Byers. N. Y., Chic. & Bost., Charles Scribners Sons (copyright 1917). 129 pp., 49 illus., 8 x 5 in., cloth. $1.00.

A companion book to the author's "Inorganic Chemistry."

INTEGRAL CALCULUS.

By H. B. Phillips. N. Y., John Wiley & Sons, Inc.; Lond., Chapman & Hall, Ltd., 1917. 194 pp., 124 illus., 7 x 5 in., cloth. $1.25.

Contents: Integration; Formulas and Methods of Integration; Definite Integrals; Simple Areas and Volumes; Other Geometrical Applications; Mechanical and Physical Applications; Approximate Methods; Double Integration; Triple Integration; Differential Equations; Supplementary Exercises, Answers, Table of Integrals, Table of Natural Logarithms; Index.

THE DEVELOPMENT OF FOREST LAW IN AMERICA:

A Historical Presentation of the Successive Enactments, by the Legislatures of the Forty-eight States of the American Union and by the Federal Congress, Directed to the Conservation and Administration of Forest Resources. By J. P. Kinney. N. Y., John Wiley & Sons, Inc.; Lond., Chapman & Hall, Ltd., 1917. 18 + 254 + 21 pp., 9 x 6 in., cloth. $2.50.

The author has sought to confine his work to a logical presentation of the chronological development of legislation that was directed to the preservation of existing forest resources, the reforestation and extension of forest areas, and the systematic management of forests for productive purposes.

VENTILATION LAWS IN THE UNITED STATES:

Also Board of Health Requirements and Regulations of National Board of Fire Underwriters. Together with Model Ventilation Requirements as Promulgated by the American Society of Heating and Ventilating Engineers. 3d rev. ed. N. Y., Heating and Ventilating Magazine Company (copyright 1917). 178 pp., 3 illus., 6 x 5 in., cloth. $1.00.

Proposed smoke prevention codes for large and small cities are included.

THE LIGHTING ART:

Its Practice and Possibilities. By M. Luckiesh. N. Y., McGraw-Hill Book Co., Inc.; Lond., Hill Publishing Co., Ltd., 1917. 9 + 229 pp., 43 illus., 9 x 6 in., cloth. $2.50.

A discussion dealing with the scientific and artistic aspects of the subject, avoiding those which are commonly discussed and omitting engineering data and considerations. The author's aim is to call attention to new viewpoints and new possibilities in the use of artificial light.

STREET RAILWAY FARES:

Their Relation to Length of Haul and Cost of Service. Report of Investigation Carried on in the Research Division of the Electrical Engineering Department of the Massachusetts Institute of Technology. By Dugald C. Jackson and David J. McGrath. (Research Division *Bulletin No. 14*). N. Y., McGraw-Hill Book Company, Inc.; Lond., Hill Publishing Company, Ltd., 1917. 13 + 169 pp., 58 illus., 9 x 6 in., cloth. $2.50.

Gives the data collected and conclusions reached in a research on the economics of the street railway industry with particular reference to the adequacy of the five-cent fare under present conditions and the length of ride given for this fare.

THE ENGINEER'S MANUAL.

By Ralph G. Hudson, Assisted by Joseph Lipka, Howard B. Luther, and Dean Peabody. N. Y., John Wiley & Sons, Inc.; Lond., Chapman & Hall, Ltd., 1917. 315 pp., 227 illus., 8 x 5 in., cloth. $2.00.

This work originated from the conception that the practising engineer or engineering student would welcome a consolidation of the formulas and constants for which he is accustomed to search through several volumes, and that the application of each formula might be explained more concisely than in texts devoted exclusively to the process of derivation. With this in view, those engineering formulas, mathematical operations, and tables of constants which appear to be most useful are presented in systematic order and in size of book designed to fit the pocket. Each formula is preceded by a statement in which its application, the symbology of the involved physical quantities, and definite units of measurement are indicated.

BUILDING STONES AND CLAYS:

A Handbook for Architects and Engineers. By Charles H. Richardson. Syracuse, The Author, The Syracuse University Book Store, 1917. 14 + 437 pp., 313 illus., 9 x 6 in., cloth. $5.50.

The object of this volume has been to furnish an elementary knowledge of the essential minerals in building stones and the objectionable minerals they sometimes contain; to show the chief characteristics of the more important building stones; to give their geographical distribution and range in compressive strength; to impart some information as to the physical and chemical properties of clays and the products that may be manufactured from them. References are given with each chapter, and a glossary is appended to the work.

TECHNIC OF SURVEYING INSTRUMENTS AND METHODS;

Including General and Detailed Instructions for Field and Office Work of Extended Students' Surveys. By Walter Loring Webb and John Charles Lounsbury Fish. N. Y., John Wiley & Sons, Inc.; Lond., Chapman & Hall, Ltd., 1917. 16 + 319 pp., 59 illus., 7 x 4 in., leather. $2.00.

A combination of the greater part of Webb's "Engineering Instruments", a rewritten text of Fish's "Technic of Surveying Instruments", and considerable additional material. It is intended to supplement the general directions given in textbooks on surveying by supplying detailed directions for specific operations in the field and office.

DIE FISCHWEGE AN WEHREN UND WASSERWERKEN IN DER SCHWEIZ.

Von A. Härry. (Publikationen des Schweizerischen Wasserwirtschafts-Verbandes Nr. 5). Zürich und Leipzig, Rascher & Co., 1917. 115 pp., 101 illus., 12 x 9 in., pap. 4 francs. (Gift of Schweizerischen Wasserwirtschafts-Verbandes).

A systematic and detailed report on the installation of fishways in water-power plants, their cost, and methods of construction, with a study of their value as a means of conserving the fish supply of the country. Numerous drawings and photographs of existing installations are given.

ROLL OF HONOR

A List of Members of the Society Who are Serving in the Army or Navy of the United States or Any of Its Allies.*

Abbot, Frederic V. Brig.-Gen., Corps of Engrs., N. A., Office, Chief of Engrs., U. S. A., Washington, D. C.

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Ackerman, Alexander S. 1st Lieut., E. O. R. C., Care, Depot Q. M., New York City.

Adams, Edward M. Maj., U. S. A., Care, The Adjutant-Gen., U. S. A., Washington, D. C.

Adams, Milton Jewell. Capt., Co. C, 114th Engrs., Camp Beauregard, Alexandria, La.

Albert, Frederick W. Capt., E. O. R. C., 23 Isham St., Burlington, Vt.

Alden, Herbert C. 1st Lieut., C. A. C., N. G. U. S., Fort Schuyler, New York City.

Allen, Franklin R. Capt., E. O. R. C., 3d Co., Fort Leavenworth, Kans.

Allen, Ralph B. 1st Lieut., Co. B, 25th Engrs., Am. Exp. Force.

Allen, Walter Hinds. Civ. Engr., U. S. N. (rank of Lt.-Commander); Public Works Officer, Navy Yard, New York City.

Allison, William F. Maj., E. O. R. C., Am. Exp. Force, France.

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Andrews, Clarence R. Capt., E. O. R. C., 1st Co., Camp American University, Washington, D. C.

Andrews, J. H. M. Maj., 103d Engrs., Camp Hancock, Augusta, Ga.

Angas, Robert M. 1st Lieut., 106th Engrs., Camp Wheeler, Macon, Ga.

Ardery, Edward Dahl. Maj., Corps of Engrs., U. S. A., Am. Exp. Force.

Armitage, George W. Capt., Q. M. R. C., Honolulu, Hawaii.

Armstrong, Charles Johnstone. Brig.-Gen.; Chf. Engr., Canadian Army Corps, B. E. F., 604 Royal Trust Bldg., Montreal, Que., Canada.

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* This list is made up from replies to a circular forwarded to members of the Society, and others, and is probably neither accurate nor complete. It is requested that the attention of the Secretary be called to any omissions or inaccuracies in order that they may be corrected in subsequent lists.

Armstrong, Merwin. Capt., Co. D, 105th Engrs., Camp Sevier, Greenville, S. C.

Arn, William G. Capt.; Adjutant, 1st Bn., 13th U. S. Engrs. (Ry.), Am. Exp. Force, France.

Asplundh, E. T. Capt.; Supply Officer, 103d Engrs., 28th Div., Camp Hancock, Augusta, Ga.

Atterbury, W. W. Brig.-Gen.; Director Gen. of Transportation, Am. Exp. Force.

Austill, Hurieosco. Capt., E. O. R. C., Co. D, 501st Engrs., Service Bn., Am. Exp. Force.

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Bailhache, John G. 2d Lieut., E. O. R. C., Vancouver Barracks, Vancouver, Wash.

Bakenhus, R. E. Civ. Engr., U. S. N. (rank of Commander); Bureau of Yards and Docks, Washington, D. C.

Baker, Albert A. Civ. Engr., U. S. N. (rank of Lt.-Commander); Naval Training Station, Gulfport, Miss.

Baker, H. S. Capt., E. O. R. C.; Const. Q. M., Camp Bowie, Fort Worth, Tex.

Balch, William H. Capt., E. O. R. C., Camp American University, Washington, D. C.

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Bartholomew, B. W. 2d Lieut., E. O. R. C., 301st Engrs., Camp Devens, Ayer, Mass.

Battie, H. S. 1st Lieut., E. O. R. C., 1st Co., Fort Leavenworth, Kans.

Baxter, O. G. Capt., E. O. R. C., Camp Pike, Little Rock, Ark.

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Beeman, Thomas R. Capt., E. O. R. C., Am. Exp. Force, France.

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Begg, R. B. H. Capt., E. O. R. C., Am. Exp. Force, France.

Behrman, I. E. 1st Lieut., E. O. R. C., 26th Engrs., Am. Exp. Force, France.

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Black, James B. 1st Lieut., E. O. R. C., Chf. Engr.'s Office, Headquarters, Am. Exp. Force, France.

Black, Roger Derby. Lt.-Col., Corps of Engrs., U. S. A., Headquarters, Am. Exp. Force.

Black, R. P. 1st Lieut., E. O. R. C. (*Unassigned*), Sewanee, Tenn.

Black, William M. Brig.-Gen.; Chf. of Engrs., U. S. A., War Dept., Washington, D. C.

Blackman, J. W. B. Lieut., Canadian Ry. Troops, Canadian Expeditionary Force.

Blair, McCrea Parker. Capt., No. 1 Depot Bn., Manitoba Reg., Winnipeg, Man., Canada.

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Boggs, F. C. Lt.-Col., Corps of Engrs., U. S. A.; Col., 315th Engrs., N. A., Camp Travis, San Antonio, Tex.

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Bolin, Harry W. Sergeant, Co. E, 23d Engrs., Camp Meade, Annapolis Junction, Md.

Bond, P. S. Col., 107th Engrs.; Div. Engr., 32d Div., Camp MacArthur, Waco, Tex.

Boorman, Kitchell M. Sapper, Canadian Engrs., Training Depot, St. John, Que., Canada.

Booz, Horace Corey. Maj., Staff of Director-Gen. of Rys., Am. Exp. Force, France.

Bott, C. N. 1st Lieut., E. O. R. C., 315th Engrs., Camp Pike, Little Rock, Ark.

Bowlby, Henry L. Capt., 20th Engrs.; Regimental Adjt., Camp American University, Washington, D. C.

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Brown, Elliott C. Lt.-Commander, U. S. N. R., Naval Base, Norfolk, Va.

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Buck, Richard S. Maj., 11th Engrs. (Ry.), Am. Exp. Force, France.

Buck, Walter Van. Capt., 23d Engrs., Camp Meade, Annapolis Junction, Md.

Buckwalter, Harris D. Capt., E. O. R. C., 1st Co., Camp American University, Washington, D. C.

Bunker, Stephen S. Capt., E. O. R. C., Co. C, 503d Service Bn., Camp Merritt, Tenafly, N. J.

Burdett, O. L. Capt., Co. A, 25th Engrs., Camp Devens, Ayer, Mass.

Burger, Alfred A. Corporal, 34th Co., 9th Training Bn., 158th Depot Brigade, Camp Sherman, Chillicothe, Ohio.

Burgess, Harry. Col., 16th Engrs., Am. Exp. Force, France.

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Carey, William N. Capt. and Adjt., 313th Engrs., Camp Dodge, Des Moines, Iowa.

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Cattell, William A. Maj., E. O. R. C. (*Unassigned*), Foxcroft Bldg., San Francisco, Cal.

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Chittenden, Albert Frederick. 1st Lieut., Co. F, 18th Engrs. (Ry.), Am. Exp. Force, France.

Christensen, George A. Capt. (Q. M.), E. O. R. C., Camp Kearney, San Diego, Cal.

Christophers, Reginald Gillon. No. 60286, 2d Lieut., 34th Reinforcements, New Zealand Exp. Forces, Care, G. P. O., Wellington, New Zealand.

Churchill, Percival M. Maj., E. O. R. C., Commanding 2d Bn., 304th Engrs., Camp Meade, Baltimore, Md.

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Conrad, Clarence Knight. Maj., E. O. R. C.; Constr. Q. M., Metuchen, N. J.

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Strecker, R. A. Capt., E. O. R. C., Co. A, 309th Engrs., Camp Zachary Taylor, Louisville, Ky.

Street, J. Z. Private, Co. E., 308th Engrs., Camp Sherman, Chillicothe, Ohio.

Strickler, F. W. Capt., E. O. R. C. (*Unassigned*); Dist. Engr., Erie R. R. Co., Room 301, K. of C. Bldg., Youngstown, Ohio.

Strickler, T. J. Capt., E. O. R. C., Fort Leavenworth, Kans.

Stroebe, George G. Capt., N. A., Philippine Islands.

Sturtevant, C. W. Lt.-Col., 15th U. S. Engrs., Am. Exp. Force, France.

Summers, R. E. J. 1st Lieut., E. O. R. C., Co. F, 16th U. S. Engrs. (Ry.), Am. Exp. Force.

Suter, Russell. Capt., E. O. R. C., Am. Exp. Force, France.

Sutton, Frank. Maj., E. O. R. C., 2d Bn., 25th Engrs., Camp Devens, Ayer, Mass.

Swaren, J. W. Capt., E. O. R. C.; Engr. Instructor, E. O. T. C., Vancouver Barracks, Vancouver, Wash.

Sweeney, Harry C. Capt., Q. M. R. C., Care, Office of Cantonment Constr., 15th and M Sts., Washington, D. C.

Sykes, George. Capt., E. O. R. C. (*Unattached*), Am. Exp. Force, France.

Tainter, F. S. Maj., E. O. R. C., 60 Wall St., New York City.

Tate, R. L. 1st Lieut., E. O. R. C., 303d Engrs., Camp Dix, Wrightstown, N. J.

Taylor, Harry. Brig.-Gen., N. A.; Chf. Engr. Officer, A. E. F., Headquarters, Am. Exp. Force, France.

Taylor, Henry. Capt., E. O. R. C., 304th Engrs., Camp Meade, Baltimore, Md.

Taylor, William T. Capt., Royal Flying Corps, France.

Ten Hagen, Henry. 2d Lieut., E. O. R. C., 303d Engrs., Camp Dix, Wrightstown, N. J.

Tenney, Willis R. Capt., 317th Engrs., Camp Sherman, Chillicothe, Ohio.

Thomas, Charles D. Capt., E. O. R. C., Co. B, 507th Engrs., Camp Travis, San Antonio, Tex.

Thomsen, S. L. Capt., E. O. R. C., Gilman Apartment D-1, 31st and Calvert Sts., Baltimore, Md.

Throop, George H. Capt., E. O. R. C., Co. E, 24th Engrs., Camp Dix, Wrightstown, N. J.

Thurber, Clinton D. Civ. Engr., U. S. N. (rank of Lt.-Commander); Bureau of Yards and Docks, Washington, D. C.

Thurston, Eugene T. Capt., E. O. R. C., Vancouver Barracks, Vancouver, Wash.

Tilden, Charles Joseph. Capt., E. O. R. C. (*Unassigned*), Care, Johns Hopkins Univ., Baltimore, Md.

Todd, Frank Herbert. Maj., Q. M. R. C., in Chg. of Utilities, Camp Travis, San Antonio, Tex.

Tolles, F. C. Private, Engr. Corps, Ohio Div., Camp Sheridan, Montgomery, Ala.

Topping, Perry. Capt., E. O. R. C., Fort Leavenworth, Kans.

Townsend, C. McD. Col., Corps of Engrs., U. S. A., 12th Engrs. (Ry.), Am. Exp. Force, France.

Trout, Alex Linn. Capt., E. O. R. C., 2d Co., Fort Leavenworth, Kans.

True, Albert O. Capt., E. O. R. C. (*Unassigned*), High St., Rensselaer, N. Y.

Trueblood, P. McG. Lieut., U. S. N. R. F., U. S. S. *Wm. Rockefeller,* Care, Postmaster, New York City.

Tucker, H. F. Ensign, U. S. N. R. F., Seattle Construction & Dry Dock Co., Seattle, Wash.

Turley, Jay. Capt., E. O. R. C., 316th Engrs., 91st Div., Camp Lewis, Tacoma, Wash.

Turner, Daniel Norman. 2d Lieut., E. O. R. C., 304th Engrs., Camp Meade, Baltimore, Md.

Turner, Nathaniel Parker. Capt.; Topographical Officer, 111th Engrs., Camp Bowie, Fort Worth, Tex.

Tuska, Gustave R. Maj., E. O. R. C., 68 William St., New York City.

Tyson, William C. Capt., E. O. R. C., 6th Co., Fort Leavenworth, Kans.

Van Buren, Maurice Pelham. 2d Lieut., Corps of Engrs., U. S. A., New Brighton, N. Y.

Vandemoer, N. C. Capt., E. O. R. C., 1113 East 13th Ave., Denver, Colo.

Vandevanter, Elliott. Capt., Co. D, 503d Engrs., Camp Merritt, Tenafly, N. J.

Vandevoort, B. F. Capt., E. O. R. C., 86 Houston Ave., Middletown, N. Y.

Van Ness, R. A. Lieut., E. O. R. C., 4th Co., Fort Leavenworth, Kans.

Van Pelt, Sutton. Capt., E. O. R. C., Fort Leavenworth, Kans.

Van Suetendael, Achille O. Capt., E. O. R. C., 1419 F St., N. W., Washington, D. C.

Vincent, W. H. 1st Lieut., E. O. R. C., 125 French Ave., Winchester, Ky.

Wade, George W. 1st Lieut., Co. D, 2d Bn., 117th Engrs., 42d Div., Am. Exp. Force, France.

Wadsworth, George R. Maj., Signal Corps, U. S. A.; Chf. Engr., Naval Aircraft Factory, Navy Yard, Philadelphia, Pa.

Wagner, John, Jr. 1st Lieut., Cavalry U. S. R., Am. Exp. Force.

Waite, Clement F. 2d Lieut., E. O. R. C., Vancouver Barracks, Vancouver, Wash.

Walker, Elton D. Capt., E. O. R. C., Co. A, 5th U. S. Engrs., Am. Exp. Force, France.

Walker, Harry Bruce. Capt., E. O. R. C., Fort Leavenworth, Kans.

Walker, Meriwether Lewis. Col., Corps of Engrs., U. S. A., 116th Engrs., 41st Div., Camp Mills, Hempstead, N. Y.

Walker, W. Kemp. Capt., E. O. R. C. (*Unassigned*), Montpelier, Ohio.

Walton, H. B. Capt., E. O. R. C., 312th Engrs., Camp Pike, Little Rock, Ark.

Ward, Jasper D. Corporal, Co. C, 55th Inf., Chickamauga Park, Chattanooga, Tenn.

Ward, Lyman Wise. Capt., C. A., U. S. R., 5th Co., Fort Warden, Wash.

Ware, John. 1st Lieut., Co. A, 101st Engrs., Am. Exp. Force, France.

Ware, Norton. Capt., E. O. R. C., Vancouver Barracks, Vancouver, Wash.

Warfield, Ralph M. Civ. Engr., U. S. N. (rank of Lt.-Commander); Navy Aeronautic Station, Pensacola, Fla.

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Warner, Elwin S. Capt., E. O. R. C., Co. B, 301st Engrs., Camp Devens, Ayer, Mass.

Warren, James G. Col., Corps of Engrs., U. S. A., 540 Federal Bldg., Buffalo, N. Y.

Warren, Philip R. Lieut., Royal Engrs., British Army, Richborough, Kent, England.

Washington, W. O. Capt., E. O. R. C. (*Unassigned*), San Antonio, Tex.

Watkins, Guy A. Capt., E. O. R. C., Fort Sill, Okla.

Watson, George L. Capt., E. O. R. C., Co. B, 30th U. S. Engrs., Headquarters, Washington, D. C.

Watson, Winslow B. Lieut., Co. C, 7th N. Y. Inf. (114th), Camp Wadsworth, Spartanburg, S. C.

Watt, David A. Maj., E. O. R. C., Office of Chf. of Engrs., Washington, D. C.

Waugh, William H. Capt., E. O. R. C.; Engr. Officer, Alaska Road Comm., Juneau, Alaska.

Way, W. F. Sergeant, Co. D, 18th Engrs. (Ry.), U. S. Army Post Office No. 705, Am. Exp. Force, France.

Weaver, C. J. 1st Lieut., E. O. R. C. (*Unassigned*), Waterford, N. Y.

Weaver, Frank L. 1st Lieut., 305th Engrs., Camp Lee, Petersburg, Va.

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Webb, I. Gano. Capt., E. O. R. C., Care, Harbor Comm., Ferry Bldg., San Francisco, Cal.

Webb, Walter Loring. Maj., E. O. R. C., Am. Exp. Force.

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Wenige, Arthur E. Maj., E. O. R. C., 317th Engrs., Camp Sherman, Chillicothe, Ohio.

Wenzell, Andrew P. Capt., Co. F, 16th U. S. Engrs. (Ry.), Am. Exp. Force, France.

West, Edward Hazzard. Capt., E. O. R. C., Fort Leavenworth, Kans.

Wheat, G. Neville. Capt., E. O. R. C., Fort Leavenworth, Kans.

Whiteaker, Robert O. Capt., Troop C, 1st Tex. Cav., Camp Bowie, near Fort Worth, Tex.

Whitman, Ezra B. Maj., Q. M. R. C.; Chf. Engr. of Utilities, Camp Meade, Admiral, Md.

Whitman, Ralph. Civ. Engr., U. S. N. (rank of Lt.-Commander); Aid on Staff of U. S. Military Governor of Santo Domingo, Santo Domingo, Dominican Republic.

Whitney, Ralph Edward. 1st Lieut., Sanitary Corps, N. A., Camp Greenleaf, Fort Oglethorpe, Ga.

Wickersham, John H. Capt., E. O. R. C., Engrs.' Constr. Gang, Phœnix Constr. Co., Am. Exp. Force, France.

Wiggin, Thomas H. Capt., E. O. R. C., Care, Gen. Taylor, Am. Exp. Force, France.

Wild, H. J. Capt., E. O. R. C., War Dept., Washington, D. C.

Wilgus, W. J. Maj., E. O. R. C.; Director Gen. of Transportation, Headquarters, Am. Exp. Force, France.

Willard, N. R. Capt., E. O. R. C., 3d Co. Camp American University, Washington, D. C.

Williams, Alan F. Headquarters Co., 18th Engrs. (Ry.), Am. Exp. Force, France.

Williams, Gardner S. Maj., E. O. R. C., Camp Beauregard, Alexandria, La.

Williar, Harry D., Jr. 1st Lieut., Co. E, U. S. Engrs., Am. Exp. Force, France.

Willis, Walter John. Lieut. (EDO), Bldg. 22, Navy Yard, Brooklyn, N. Y.

Wilson, Everitt W. Capt., E. O. R. C., Camp American University, Washington, D. C.

Wilson, Harry P. Capt., E. O. R. C., Fort Leavenworth, Kans.

Wilson, Robert B. M. Capt., E. O. R. C., 311th Engrs., Camp Grant, Rockford, Ill.

Wing, Charles B. Maj., 23d Engrs., Camp Meade, Admiral, Md.

Winn, Walter E. Maj., 114th Engrs (Pioneer Regt.), Camp Beauregard, Alexandria, La.

Winn, Walter S. Maj., E. O. R. C., 4th and 1st National Bank Bldg., Nashville, Tenn.

Winton, Walter Ferrell. Capt., 14th U. S. Field Artillery, Fort Sill, Okla.

Wood, Frederic J. Maj., E. O. R. C., 15th, cor., M Sts., N. W., Washington, D. C.

Woodard, Wilkie. Capt., Co. H, 35th Engrs., Camp Grant, Rockford, Ill.

Woodle, Bernon Tisdale. Capt., 305th Engrs., Care, Depot Engr., Coca Cola Bldg., Baltimore, Md.

Woolworth, W. H. 1st Lieut., 28th U. S. Infantry; A. D. C., 2d Brigade, 1st Div., Am. Exp. Force, France.

Wright, John Bertram. Capt., E. O. R. C., P. O. Box 411, Oxford, Mass.

Wrightson, William D. Maj.; Chf. of San. Corps, Surgeon-General's Office, War Dept., Washington, D. C.

Yereance, Alex. W. Lieut., Co. F., 305th Engrs., Camp Lee, Petersburg, Va.

Yost, Howard McC. Capt., E. O. R. C., 2030 F St., N. W., Washington, D. C.

Young, Samuel M. Capt., E. O. R. C. (*Unassigned*), 635 Common St., New Orleans, La.

Zinn, George A. Col., Corps of Engrs., U. S. A., 321 Custom House, Portland, Ore.

MEMBERSHIP

(From November 9th to December 6th, 1917)

ADDITIONS

MEMBERS		Date of Membership.
BROWN, CHARLES FRANKLIN. Cons. Engr. (Brown & Kleinschmidt, Inc.), 306 Dooly Block, Salt Lake City, Utah	Assoc. M. M.	Oct. 7, 1908 June 12, 1917
CLOGHER, ALEXANDER CLARKE. Hydr. Engr., Elec. Bond & Share Co., 71 Broadway, New York City		Nov. 27, 1917
DURHAM, ROBERT PAUL. With Macdonald Eng. Co., 949 Monadnock Bldg., Chicago, Ill		Nov. 27, 1917
FLEMING, THOMAS, JR. Hydr. and San. Engr., Union Bank Bldg., Pittsburgh, Pa	Jun. Assoc. M. M.	Nov. 5, 1907 Oct. 5, 1909 Nov. 27, 1917
HINKLE, ALBERT HARRISON. Deputy Highway Commr., Bureau of Maintenance and Repair, 1722 Summit St., Columbus, Ohio	Assoc. M. M.	Oct. 1, 1912 Nov. 27, 1917
JOHNSON, DAVID CLAYTON. Engr., The National City Co., 55 Wall St., New York City (Res., 1140 President St., Brooklyn, N. Y.)	Jun. Assoc. M. M.	Sept. 1, 1908 May 7, 1913 Nov. 27, 1917
KLUEGEL, HARRY ALLARDT. Engineers Club, 57 Post St., San Francisco, Cal		May 15, 1917
MANSFIELD, ROYAL JOHN. Cons. Engr., 135 William St., New York City	Jun. Assoc. M. M.	Dec. 2, 1902 July 10, 1907 Nov. 27, 1917
PEOTTER, REUBEN SYLVESTER. Managing Director, Surinaamsche Bauxite Maatschappij and Demerara Bauxite Co., Ltd., Paramaribo, Dutch Guiana	Assoc. M. M.	Nov. 30, 1909 Oct. 9, 1917
PERKINS, SETH. Engr. and Contr. (Seth Perkins & Sons), St. Augustine, Fla		Sept. 11, 1917
PURVER, GEORGE MOSES. Cons. Engr., 1056 Dean St., Brooklyn, N. Y	Assoc. M. M.	Oct. 3, 1911 Oct. 9, 1917
RODMAN, GEORGE EDWARD. Asst. Engr., Dept. of Water Supply, 317 West 99th St., N. Y. City		Nov. 27, 1917
STABLER, HERMAN. Chf. Engr., Land Classification Board, U. S. Geological Survey, Washington, D. C	Assoc. M. M.	Nov. 1, 1910 Nov. 27, 1917
STONE, SOLON JONES. Supt., The John W. Cowper Co., Inc., Garden City, N. Y		Nov. 27, 1917
TURNER, NATHANIEL PARKER. Capt., 111th Engrs.; Topographical Officer, Camp Bowie, Fort Worth, Tex	Assoc. M. M.	Dec. 1, 1908 Sept. 11, 1917

MEMBERS (*Continued*) — Date of Membership.

VOETSCH, CARL. Hydr. and Elec. Engr., Utica Gas & Elec. Co., 1225 Kemble St., Utica, N. Y. .... Nov. 27, 1917

ASSOCIATE MEMBERS

ACKER, ROBERT LOUIS. Supt. of Constr., H. L. Stevens & Co., 910 South Michigan Ave., Chicago, Ill. .... Nov. 27, 1917

ALLEN, HENRY STANLEY. 3533 Tracy Ave., Kansas City, Mo. .... Oct. 9, 1917

BILDERBECK, GEORGE LESLIE. Asst. Engr., Rivers, Harbors and Bridge Comm., Groton, Conn. .... Nov. 27, 1917

BITHER, TOM ALLEN. Lieut., Aviation Section, S. R. C., U. S. A., Aviation School, San Diego (Res., 1630 Josephine St., Berkeley), Cal. .... Jun. Oct. 1, 1913; Assoc. M. Oct. 9, 1917

BLISS, TULLA ETHAN. Asst. Engr., St. L., S. F. Ry., 1124 Sledge, Memphis, Tenn. .... Sept. 11, 1917

BOYNTON, HERBERT LESLIE. 16 Sixth Ave., Haverhill, Mass. Nov. 27, 1917

BROKER, ALBERT EDWARD. Engr., Portland Cement Assoc., 403 Farmers Bank Bldg. (Res., 6425 Bartlett St.), Pittsburgh, Pa. Jun. Jan. 17, 1916; Assoc. M. Nov. 27, 1917

BRYAN, GEORGE, JR. Sales Engr., Am. Bridge Co., 204 Wolvin Bldg., Duluth, Minn. .... Jun. Jan. 3, 1911; Assoc. M. Nov. 27, 1917

BUTLER, WILLIAM PARKER. Army and Navy Club, Washington, D. C. .... Nov. 27, 1917

CAMPBELL, THOMAS FRANCIS. Asst. Engr., Street Line and Street Layout Dept., City Engr.'s Dept., Providence, R. I. .... Nov. 27, 1917

CARSON, MERWIN BISHOP. Res. Engr., Representing Wm. T. Donnelly of New York and The Inter-Island Steam Navigation Co. of Honolulu, Honolulu, Hawaii ..... Oct. 9, 1917

CASEY, ISAAC JACQUES, JR. Town Engr., Town Hall, Irvington, N. J. .... June 11, 1917

CROLL, HERBERT GREISS. 23 Union Ave., Crafton, Pa. .... Jun. Nov. 12, 1913; Assoc. M. Sept. 11, 1917

CROWLEY, JOHN ADAM. Structural Engr., Butterworth Judson Corporation, 40 Oakland Ave., Lynbrook, N. Y. Nov. 27, 1917

DANFORTH, GEORGE CLAPP. Capt., E. O. R. C., 29 Pleasant St., Gardiner, Me. .... Nov. 27, 1917

FAIRLIE, JOHN WALTER. 260 West 57th St., New York City. Sept. 11, 1917

FEWSMITH, WILLIAM LEE. Eastern Sales Mgr., Robins Conveying Belt Co., 73 High St., Glen Ridge, N. J. ...... Nov. 27, 1917

FOARD, ARTHUR VIRDIN. Asst. Supt., Maryland Dredging & Contr. Co., Aberdeen, Md. .... Oct. 9, 1917

GABELMAN, CHARLES GROVER. Care, Bureau of Public Works, Manila, Philippine Islands .... May 15, 1917

ASSOCIATE MEMBERS (*Continued*)

		Date of Membership.
GATENS, RAY STUART. With Continental Public Works Co., 30 Church St., Room 418 E, New York City		Nov. 27, 1917
GODDARD, LESLIE DREW. Capt., E. O. R. C., 144 Alexandrine Ave., W., Detroit, Mich		May 15, 1917
GOODFELLOW, JAMES GORDON. Capt., R. E., 49 Arden St., Edinburgh, Scotland	Jun. Assoc. M.	Sept. 3, 1913 Oct. 9, 1917
GURNEY, LESTER. Asst. Engr., Dept. of Streets and Eng., Municipal Bldg., Springfield, Mass.	Jun. Assoc. M.	Dec. 5, 1911 Nov. 27, 1917
HANSEN, JOHN MILTON. City Engr., Box 532, Jamestown, N. Dak		Nov. 27, 1917
HELMSTETTER, GEORGE ALFRED. Park Engr., Syracuse Park Comm., 701 Teall Ave., Syracuse, N. Y	Jun. Assoc. M.	Aug. 31, 1915 Nov. 27, 1917
HENDERSON, ALFRED REID. 187 Bergen Ave., Jersey City, N. J		Nov. 27, 1917
HOAR, ALLEN. Civ. Engr., U. S. N. R. F., Navy Yard, Mare Island, Cal	Jun. Assoc. M.	Feb. 4, 1914 Oct. 9, 1917
HOLSTLAW, CHARLES HENRY. 1st Lieut., 124th Infantry, Camp Wheeler, Macon, Ga		Oct. 9, 1917
HOWELL, CLARENCE SCOTT. 550 West 174th St., New York City		Nov. 27, 1917
JOHNTZ, ALBERT FREDERICK. Asst. Engr., The Cuba R. R., Dept. de Via, Camaguey, Cuba		Oct. 9, 1917
KALBERG, SWAN AUGUST. Structural Engr., Fred T. Ley & Co., Inc., 112 Magnolia Terrace, Springfield, Mass.		Nov. 27, 1917
KILLOUGH, EDWARD MATHIAS. Junior Structural Engr., Interstate Commerce Comm., Div. of Valuation, 116 Market St., Bethlehem, Pa		Nov. 27, 1917
LEON, HARRY MCCONNELL. Junior Asst. Engr., Public Service Comm., New York City, 953 Elmore Pl., Brooklyn, N. Y		Nov. 27, 1917
LEWALD, ALFRED. Supt., The Moreno-Burkman Constr. Co., 4255 Athlone Ave., St. Louis, Mo		Nov. 27, 1917
LUMSDEN, HUGH JOHN. Detail Draftsman, Structural Div., City Engr.'s Office, 2301 Ward Ave., Kansas City, Mo.		Oct. 9, 1917
MACLEAN, CHARLES EDWARD. Watermaster, U. S. Reclamation Service, Boise, Idaho		June 11, 1917
MCDONALD, WILLIAM FREDERICK. Asst. Engr., C., M. & St. P. Ry., 1120 Wells St., Apartment 8, Milwaukee, Wis.		Oct. 9, 1917
MEAD, ROYAL LEE. 10 Pearson St., Chicago, Ill		Nov. 27, 1917
MOELLER, HENRY LEWIS. Chf. Engr., Martini & Huneke Co. of America, 1201 Hudson St., Hoboken (Res., 20 Belleview Ave., Highwood Park, Weehawken), N. J		Nov. 27, 1917

ASSOCIATE MEMBERS (*Continued*)

		Date of Membership.
MULLEN, CHARLES AUGUSTINE. Director, Paving Dept., Milton Hersey Co., Ltd., 84 St. Antoine St., Montreal, Que., Canada		Nov. 27, 1917
NEWKIRK, SAMUEL FRANK, JR. Lamartine Ave., Bayside, N. Y.	Jun. Assoc. M.	Feb. 4, 1914 Nov. 27, 1917
OXER, GEORGE CARROL. 136 West 73d St., New York City		Nov. 27, 1917
PEARSON, HARRY BROWNLEY, JR. Res. Engr., Julian Kennedy, Care, The Alan Wood Iron & Steel Co., Swedeland, Pa.	Jun. Assoc. M.	Oct. 7, 1914 Nov. 27, 1917
PERKINS, SETH, JR. 2d Lieut., Field Artillery, U. S. R., 11th Field Artillery, Douglas, Ariz.		Nov. 27, 1917
PITCHER, FLOYD JACOB. Asst. to E. W. Wiggin, 113 Church St., New Haven, Conn.		Nov. 27, 1917
SARTZ, JACOB PETER. Chf. Structural Engr., Standard Oil Co. of New York, Shanghai, China		Sept. 11, 1917
SCUDDER, HENRY DARCY, JR. Archt. and Engr., Essex Bldg., Newark, N. J.		Nov. 27, 1917
SHAW, WILLIAM CHECKLEY, JR. Care, Div. of Valuation, Interstate Commerce Comm., Chattanooga, Tenn.		Jan. 17, 1916
SHIBLEY, KENNETH. Mgr. and Member of Firm, California Jewell Filter Co., 1218 Merchants Exchange, San Francisco, Cal.		Oct. 9, 1917
STARKWEATHER, ALFRED KENNETH. Asst. Supt., Carrington Constr. Co., 366 Van Houten Ave., Passaic, N. J.	Jun. Assoc. M.	Jan. 7, 1913 Sept. 11, 1917
STAVA, WILLIAM. Care, R. R. Comm., 833 Market St., San Francisco, Cal.	Jun. Assoc. M.	Feb. 4, 1913 Oct. 9, 1917
SUTTON, DAVID. Structural Engr., Hood Rubber Co., 15 Ashmont Rd., Waban, Mass.		Nov. 27, 1917
TAUSSIG, JOHN WRIGHT. Asst. Gen. Mgr., Raymond Concrete Pile Co., 140 Cedar St., New York City	Jun. Assoc. M.	Jan. 6, 1915 Nov. 27, 1917
VAN WINKLE, ROBERT NEEL. Director and Advisor The France Stone Co., 1800 Second National Bank Bldg., Toledo, Ohio		Nov. 27, 1917
WHITWELL, EDWARD. Technical Advisor, Aeronautics, British Ministry of Munitions, Robert Treat Hotel, Newark, N. J.		Nov. 27, 1917
WOLF, WILLIAM CHARLES. Cons. Engr.; City Engr., 304 Forest Ave., Belleville, Ill.		Nov. 27, 1917

ASSOCIATES

HEADLEY, WILLIAM THOMAS. Engr. and Pres., Headley Good Roads Co., 30th and Spruce Sts., Philadelphia, Pa.		Nov. 27, 1917

JUNIORS	Date of Membership.
BIRD, HAROLD CRUSIUS. Associate Prof., Civ. Eng., Pennsylvania Military Coll., Chester, Pa	Nov. 27, 1917
BUCHNER, HAROLD FOLLMER. 24 Pennsylvania Ave., Watsontown, Pa	Oct. 9, 1917
GAMBIER-BOUSFIELD, GEORGE EDMUND. 104 West 190th St., New York City	Nov. 27, 1917
GRAHAM, ROLAND RUSSELL. 15 East Walnut Ave., Merchantville, N. J	May 15, 1917
HOPKINS, WILLIAM TRENHOLM. 105 East 31st St., Savannah, Ga	Nov. 27, 1917
PERRY, ARTHUR FRANKLIN, JR. 2d Lieut., Coast Artillery Reserve Corps, Fort Dade, Fla	Nov. 27, 1917
REESE, RAYMOND CASTLE. Draftsman and Structural Designer, Bldg. Dept., Gen. Elec. Co., 181 Woodlawn Ave., Pittsfield, Mass	Nov. 27, 1917

CHANGES OF ADDRESS

MEMBERS

BARBER, WILLIAM DAVIS. Care, Contra Costa Co., 235 Seventh St., Richmond, Cal.

BENHAM, WEBSTER LANCE. Maj., Q. M. R. C.; Engr. Officer in Chg. of Camp Utilities, Camp Funston, Kans.

BILLIN, CHARLES EMERY. 123 West 74th St., New York City.

CHAMBERS, FRANK TAYLOR. Civ. Engr., U. S. N., Comm. on Navy Yards and Naval Stations, Washington, D. C.

COE, EDWARD KIRK. Maj., E. O. R. C., Am. Exp. Force, France.

COLE, HOWARD JUDSON. Constr. Engr., Averill Park, N. Y.

CONRAD, CLARENCE KNIGHT. Maj., E. O. R. C.; Constr. Quartermaster, Metuchen, N. J.

COX, LEONARD MARTIN. Civ. Engr., U. S. N.; Public Works Officer, U. S. Naval Training Camp, San Diego, Cal.

DEAN, BERTRAND DODD. Capt., Co. B; Adjt., 25th Engrs., Camp Devens, Ayer, Mass.

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FRY, ALFRED BROOKS. Capt., U. S. N., N. V.; Engr. Aide to Industrial Mgr., U. S. Navy Yard, Brooklyn, N. Y.

GOULD, WILLIAM TILLOTSON. Capt., 301st Engrs., Camp Devens, Ayer, Mass.

GREENFIELD, ROBERT ARTHUR. 303d Engrs., Camp Dix, Wrightstown, N. J.

HALE, HERBERT MILLER. Care, Turner Constr. Co., 244 Madison Ave., New York City.

HALE, WALTER HOVEY. Vice-Pres., Adams County Light & Power Co., 120 Warm Spring Ave., Boise, Idaho.

MEMBERS (*Continued*)

HODGE, HENRY WILSON. Col., Corps of Engrs., U. S. A.; Office of Director General of Transportation, Am. Exp. Force, France.

HOUGH, ULYSSES B. Hydr. and Mill Engr., 722 Old National Bank Bldg., Spokane, Wash.

JONAH, FRANK GILBERT. Maj., Office of Director General of Transportation, Am. Exp. Force, France.

JOYNER, FRANK HALL. Road Commr., County of Los Angeles, Box 676, R. F. D. No. 1, San Gabriel, Cal.

JUBB, SHERMAN AUGUSTUS. Superv. Engr., Bureau of Yards and Docks, Navy Dept., 57 Clifford St., San Francisco, Cal.

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(September 1st to 15th, 1917)

NOTE.—*This list is published for the purpose of placing before the members of this Society, the titles of current engineering articles, which can be referred to in any available engineering library, or can be procured by addressing the publication directly, the address and price being given wherever possible.*

LIST OF PUBLICATIONS

In the subjoined list of articles, references are given by the number prefixed to each journal in this list:

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(3) *Journal,* Franklin Inst., Philadelphia, Pa., 50c.
(4) *Journal,* Western Soc. of Engrs., Chicago, Ill., 50c.
(5) *Transactions,* Can. Soc. C. E., Montreal, Que., Canada.
(6) *Journal,* Am. Inst. Architects, Washington, D. C., 50c.
(7) *Gesundheits Ingenieur,* München, Germany.
(8) *Stevens Indicator,* Hoboken, N. J., 50c.
(9) *Industrial Management,* New York City, 25c.
(11) *Engineering* (London), W. H. Wiley, 432 Fourth Ave., New York City, 25c.
(12) *The Engineer* (London), International News Co., New York City, 35c.
(13) *Engineering News-Record,* New York City, 15c.
(15) *Railway Age Gazette,* New York City, 15c.
(16) *Engineering and Mining Journal,* New York City, 15c.
(17) *Electric Railway Journal,* New York City, 10c.
(18) *Railway Review,* Chicago, Ill., 15c.
(19) *Scientific American Supplement,* New York City, 10c.
(20) *Iron Age,* New York City, 20c.
(21) *Railway Engineer,* London, England, 1s. 2d.
(22) *Iron and Coal Trades Review,* London, England, 6d.
(23) *Railway Gazette,* London, England, 6d.
(24) *American Gas Engineering Journal,* New York City, 10c.
(25) *Railway Mechanical Engineer,* New York City, 20c.
(26) *Electrical Review,* London, England, 4d.
(27) *Electrical World,* New York City, 10c.
(28) *Journal,* New England Water-Works Assoc., Boston, Mass., $1.
(29) *Journal,* Royal Society of Arts, London, England, 6d.
(30) *Annales des Travaux Publics de Belgique,* Brussels, Belgium, 4 fr.
(31) *Annales* de l'Assoc. des Ing. Sortis des Ecoles Spéciales de Gand, Brussels, Belgium, 4 fr.
(32) *Mémoires et Compte Rendu des Travaux,* Soc. Ing. Civ. de France, Paris, France.
(33) *Le Génie Civil,* Paris, France, 1 fr.
(34) *Portefeuille Economiques des Machines,* Paris, France.
(35) *Nouvelles Annales de la Construction,* Paris, France.
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(37) *Revue de Mécanique,* Paris, France.
(38) *Revue Générale des Chemins de Fer et des Tramways,* Paris, France.
(39) *Technisches Gemeindeblatt,* Berlin, Germany, 0, 70m.
(40) *Zentralblatt der Bauverwaltung,* Berlin, Germany, 60 pfg.
(41) *Electrotechnische Zeitschrift,* Berlin, Germany.
(42) *Proceedings,* Am. Inst. Elec. Engrs., New York City, $1.
(43) *Annales des Ponts et Chaussées,* Paris, France.
(44) *Journal,* Military Service Institution, Governors Island, New York Harbor, 50c.
(45) *Coal Age,* New York City, 10c.
(46) *Scientific American,* New York City, 15c.
(47) *Mechanical Engineer,* Manchester, England, 3d.
(48) *Zeitschrift,* Verein Deutscher Ingenieure, Berlin, Germany, 1, 60m.
(49) *Zeitschrift für Bauwesen,* Berlin, Germany.
(50) *Stahl und Eisen,* Düsseldorf, Germany.
(51) *Deutsche Bauzeitung,* Berlin, Germany.
(52) *Rigasche Industrie-Zeitung,* Riga, Russia, 25 kop.
(53) *Zeitschrift,* Oesterreichischer Ingenieur und Architekten Vereines, Vienna, Austria, 70h.
(54) *Transactions,* Am. Soc. C. E., New York City, $12.
(55) *Transactions,* Am. Soc. M. E., New York City, $10.
(56) *Transactions,* Am. Inst. Min. Engrs., New York City, $6.

(57) *Colliery Guardian*, London, England, 5d.
(58) *Proceedings*, Engrs.' Soc. W. Pa., 2511 Oliver Bldg., Pittsburgh, Pa., 50c.
(59) *Proceedings*, American Water-Works Assoc., Troy, N. Y.
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(111) *Journal of Electricity, Power and Gas*, San Francisco, Cal., 25c.
(112) *Internationale Zeitschrift für Wasser-Versorgung*, Leipzig, Germany.
(113) *Proceedings*, Am. Wood Preservers' Assoc., Baltimore, Md.
(114) *Journal*, Institution of Municipal and County Engineers, London, England, 1s. 6d.
(115) *Journal*, Engrs.' Club of St. Louis, St. Louis, Mo., 35c.
(116) *Blast Furnace and Steel Plant*, Pittsburgh, Pa., 15c.

LIST OF ARTICLES

Bridges.

Defective Foundation Causes Failure of Pier.* (13) Sept. 13.

Mooring and Hoisting the Suspended Span: General Principles of Last Year's Operations were Again Followed but the Design of the Apparatus and the Program for its Use Differed in Many Important Details. A. J. Meyers and M. B. Atkinson. (96) Sept. 20.

Erection and Floating of Central Span: New Span Differs from the One that Was Lost Last Year Only in Lower End Joints—Floating Arrangements, Influence of the Tides and the General Plan of Operations.* (96) Sept. 20.

Canadian Engineering Has Triumphed at Quebec: Great Central Span of the Quebec Bridge Successfully Floated, Then Hoisted Through Thirty-four Two-foot Lifts in Two Days—With New Mooring Tackle Arrangement Ordinary Storms are not Feared Now, and There Seems to be no Reason Why Plan Should not Succeed.* (96) Sept. 20.

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No Rocker Bearings nor Steel Castings this Year: Between the Lower Lifting Girders and the Suspended Span—Lower Shoe of Roller Bearing Riveted to Girder While Upper Shoe is Riveted to Span and Key Carries the Load During Lifting.* Archibald John Meyers. (96) Sept. 20; (15) Sept. 28.
The 640 Ft. Suspended Span of Quebec Bridge Hoisted to Permanent Position. (86) Sept. 26.
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Economics of Bridge Design. J. A. L. Waddell. (Lecture before School of Eng. of the Univ. of Kansas.) (86) Sept. 26.
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Quebec Suspended Span Successfully Hung from Cantilevers.* Harry Barker. (13) Sept. 27.
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Characteristics of Small Dry Cells. C. F. Burgess. (Abstract of paper read before Am. Electrochemical Soc.) (73) Aug. 17.
Chemical Composition Versus Electrical Conductivity.* C. G. Fink. (Abstract of article in *General Electric Review*.) (73) Aug. 17.
Conversion of Light into Electrical Energy. Theodore W. Case. (Paper read before New York Elec. Soc.) (73) Aug. 17.
An Alternating Current Bridge Method of Comparing Two Fixed Inductances at Commercial Frequencies.* T. Parnell. (Abstract of paper read before Physical Soc.) (73) Aug. 17.
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Insulating Lacquers. Max Bottler. (Abstracted from *Elektrotechnische Zeitschrift*, No. 11, 1917.) (73) Aug. 24.
Direct-Current Watt-Hour Meters.* R. Ziegenberg. (Abstract of article in *Elektrotechnische Zeitschrift*.) (73) Aug. 31.
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Furnace Installation at the New Sheffield Works of Messrs. Kayser, Ellison and Company, Limited.* (22) Sept. 7.
By-Product Coke and Coking Operations.* C. J. Ramsburg and F. W. Speer, Jr. (Abstract of paper presented before the Franklin Inst. and the Philadelphia Section of Am. Soc. of Mech. Engrs.) (22) Sept. 7.
The Low Temperature Distillation of Inferior Coals.* T. F. Winmill. (22) Sept. 7.
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Coal Briquetting, with Special Reference to Anthracite. Jno. A. Yeadon. (Abstract of paper read before South Wales Inst. of Engrs.) (22) Sept. 28.

Oil Burning Ranges Demolished and Battery of Fifteen Gas Ranges Installed in Hotel Kitchen in Two Nights Without Interfering With Service.* (24) Sept. 29.

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Utilization of the Sulphur Contents of Zinc Ore. H. M. Ridge. (Abstract of paper read before the London Section of the Soc. of Chemical Industry.) (22) Aug. 17.

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Mining and Smelting Copper Ore at Kalata.* F. W. Draper. (103) Sept. 1.

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Chemical Reactions of Ore Smelting. Walther Mathesius. (Abstract of paper read before the Am. Iron and Steel Inst.) (22) Sept. 14.

Simple Methods of Finding Density and Weight of Solids in Mill-Pulp.* R. B. Kiliani. (103) Sept. 15.

Cyaniding Gold Copper Ores at Pilgrims Rest. Robert Lindsay. (Abstract of paper published in *Journal* of the Chemical, Metallurgical and Min. Soc. of South Africa, April, 1917.) (16) Sept. 15.

Rôle of Manganese in Alloy Steels. Henry M. Howe. (Abstract of paper read before Am. Soc. for Testing Materials.) (16) Sept. 15.

Zinc Burning as a Metallurgical Process.* W. R. Ingalls. (16) Sept. 15.

Cyanidation of Flotation Concentrate. James G. Parmelee. (103) Sept. 15.

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The Eggertz Test for Combined Carbon in Steel. J. H. Whiteley. (Abstract of Carnegie Research Report.) (22) Sept. 21.

The Microstructure of Commercially Pure Iron Between Ar3 and Ar2. W. J. Brooke and F. F. Hunting. (22) Sept. 21.

Investigation Upon a Cast of Acid Open-Hearth Steel. T. D. Morgans and F. Rogers. (22) Sept. 21.

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Upflow Clarification of Solution.* A. W. Allen. (103) Sept. 22.
Commercial Alloy Steels: Historical and Practical Notes on Their Development and Use. Edgar D. Rogers. (19) Sept. 22.
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The Panoramic Sight.* W. L. Olmstead. (72) Serial beginning Aug. 2.
Flash Annealing Large Projectiles by Gas.* Edward A. Pilnacek. (24) Aug. 4.
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Importation of Chinese Coal Miners. P. L. Mathews. (45) Aug. 4.
Suggestions on Thin-Coal Mining.* J. F. K. Brown. (45) Aug. 4.
Reinforced Concrete: Its Application to Pit Props. W. A. Machin. (Abstract of paper read before Midland Branch of the National Assoc. of Colliery Managers.) (22) Aug. 10.
Notes on Coal Washing Experiments.* J. Pascal. (57) Aug. 10.
The Natal Coalfields—Phases in Development. Wm. Taylor Heslop. (Abstract of paper read before Chemical, Metallurgical and Min. Soc. of South Africa.) (22) Aug. 10.
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Pennsylvania Bituminous Mine Inspectors Given Life Tenure of Office. Frank Hall. (45) Aug. 11.
A Fresh Aspect of Intensive Mining in Thin Seams. George Gibb. (Abstract of paper read before Min. Inst. of Scotland.) (22) Aug. 17; (57) Aug. 17.

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Development of Deep Coal Areas in Bengal. G. George. (From *Transactions* of the Min. and Geological Inst. of India.) (57) Aug. 24; (22) Aug. 24.
The Truth About Wagon Mines. (45) Aug. 25.
The Mexican as a Coal Miner.* P. L. Mathews. (45) Aug. 25.
The Suppression of Rock Dust in Mines.* A. C. Whittome and J. H. Veasey. (From *Journal* of the South African Institution of Engrs.) (57) Aug. 31.
Double-pressure Testing of Colliery Electrical Apparatus.* L. Fokes.* (22) Aug. 31.
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Standard Reports for Accident Statistics. Albert H. Fay. (Abstract of paper read before the National Safety Council.) (16) Sept. 29.
Necessity of Mechanical Ventilation for Metal Mines. G. S. Rice. (Abstract of paper read before the National Safety Council.) (16) Sept. 29.
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Customer Ownership as a Means of Improving Public Relations. William H. Hodge. (Abstract of paper read before Graduate School of Business Administration at Harvard Univ.) (24) Sept. 1.
A Qualitative Determination of the Reflection Coefficients of Some Metals in the Schuman Region.* I. C. Gardner. (From *Astorphysical Journal*, Vol. 45, No. 1). (19) Sept. 1.
The Projection of Light. J. A. Orange. (From the *General Electric Review*.) (19) Sept. 1.
The Metric System.* C. A. Briggs. (Paper read before the National Scale Men's Assoc.) (19) Sept. 8.
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La Question du Pain en Temps de Guerre; L'Etat, le Consommateur, le Meunier, Conclusions. L. Lindet. (33) Sept. 15.

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Train Handling. G. H. Wood. (From paper presented before the Car Foremen's Assoc. of Chicago.) (15) Sept. 7.
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Factors in Locomotive Smoke Abatement: How Smoke is Formed and How Brick Arches Reduce It; Ample Combustion Space and Flameway Needed.* J. T. Anthony. (From a paper read before the Smoke Prevention Assoc.) (15) Sept. 28.
Brake Pipe Leakage and Compressor Capacity: Testing Train Leakage by Charging Through an Orifice Designed to Supply Maximum Allowable Leakage.* C. R. Weaver. (Abstract of paper presented before the Central Railway Club.) (15) Sept. 28.
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* Illustrated.

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AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

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Vol. XLIII. DECEMBER, 1917. No. 10.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

MANHATTAN ELEVATED RAILWAY IMPROVEMENTS

By F. W. Gardiner and S. Johannesson,
Members, Am. Soc. C. E.

To be Presented February 6th, 1918.

Synopsis

The work under the Manhattan Elevated Railway Improvements was authorized in 1913, and involved in general the addition of a single continuous express track, with express stations, to the Second, Third, and Ninth Avenue elevated railway lines in the City of New York, operated by the Interborough Rapid Transit Company. The work included the building of 23 miles of single-track elevated structure, the erection of about 50 000 tons of steel, the building of 638 foundations, and the construction or reconstruction of 29 stations, most of the work being in city streets often congested with traffic. The traffic on the elevated railway lines was maintained according to the regular schedule throughout the period of reconstruction.

After giving a short history of elevated railroad construction in New York City, the paper describes first the design of the steel structure, the details of which were worked out, keeping in mind the necessity of making possible the erection without interruption of

Note.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and, when finally closed, the papers, with discussion in full, will be published in *Transactions*.

traffic. Next is described the foundation work, which not only included the building of foundations in new locations, but also the removal and reconstruction of foundations under the existing structure carrying traffic. The method of erection is next described, involving in several cases partial or complete removal of the existing structure and replacing it with a new structure, and at all times maintaining traffic. The work also involved, at certain places, raising the existing structure and moving it sidewise, while trains were being operated, and the replacement of a three-span bridge across the Harlem River with a new bridge. The paper also describes the reconstruction of the stations, including the two standard types of express stations used: the "hump" type, which required the center track to be raised above the level of the local tracks so that the platforms for serving the express track could be placed above the local tracks, and the "mezzanine" type, which had two island platforms and a mezzanine station under the level of the tracks. The paper further describes the plant, the method of selecting the contractors, and the cost of the work; and, finally, an account is given of the accidents that happened during the work.

History of the Manhattan Railway Company's System.

Elevated railroad construction in New York City was commenced on July 1st, 1867, when "The West Side and Yonkers Railway Company" started to erect an experimental section on Greenwich Street between Battery Place and Cortlandt Street. This section was completed on July 2d, 1868, and was a single-track railroad. The line was extended as a single-track road through Greenwich Street and Ninth Avenue to 30th Street during the following 2 years, and was opened for traffic on February 14th, 1870. The structure was on the east side of the street through Greenwich Street and on the west side through Ninth Avenue. The trains were operated by an endless chain, driven by stationary engines below the surface of the street, at Cortlandt, Franklin, Bank, and Twenty-second Streets.

This method of traction did not prove successful, and traffic was abandoned in November, 1870, but was resumed again on April 20th, 1871, using a dummy engine, the "Pioneer", weighing 5 tons and drawing three cars.

In the fall of 1871 the bondholders of the Company organized under the General Railroad Laws as "The New York Elevated Railroad Company."

In October, 1872, the company issued a circular, in which it was stated:

"* * * We now take and receive passengers at Morris, Dey, Canal, Twelfth, and Twenty-ninth Streets. We run four unique, elegantly finished and furnished cars, made expressly for our road, capable of seating 44 passengers each, and we take no more than can be seated. We are frequently compelled to refuse passengers after our cars are full. We carry about 1 300 daily. * * * We believe we are developing what will enhance the value of real estate, solve the problem of quick transit, relieve our over-crowded streets and sidewalks, be of great public service, and a successful paying enterprise. * * *"

During the following years, the line was gradually extended northward on Ninth Avenue, in 1873 to 34th Street, in 1875 to 42d Street, and in 1876 to 59th Street. In 1877 a double-track extension was built to South Ferry, and during 1878 a double track was completed and opened for traffic over the whole line. At the same time the rolling stock was increased, consisting, in 1874, of 10 cars and 6 engines and, in 1876, of 21 cars and 15 engines.

The original structure on the east side of Greenwich Street and on the west side of Ninth Avenue between Battery Place and 59th Street was torn down in sections and replaced by a new structure, which was completed in May, 1880.

In October, 1877, the New York Elevated Railroad Company commenced the construction of the Third Avenue Line, and opened it for traffic on August 26th, 1878, between South Ferry and 42d Street. The line was extended and opened for traffic to 129th Street before the end of the year. The 42d Street branch to Grand Central Station was also opened in August, 1878; the City Hall branch, from Chatham Square to City Hall, was completed in March, 1879, and the 34th Street branch to the East River was opened in July, 1880.

Another company, the Metropolitan Railway Company, formerly called the Gilbert Elevated Railway Company, in the spring of 1876 commenced to construct what is now called the Sixth Avenue Line, between Morris Street and 59th Street, but became involved in legal difficulties, and the road was not opened until June, 1878. The same

company opened the line through 53d Street from Sixth to Ninth Avenues and the line in Ninth and Eighth Avenues from 59th to 155th Streets in 1878.

The Metropolitan Elevated Railway Company also constructed an east-side line from Chatham Square through Division Street, Allen Street, and First Avenue to 19th Street, which was opened in September, 1879; the remainder of the line—which is now the Second Avenue Line—to 129th Street, was opened in 1880.

In May, 1879, the New York Elevated Railroad Company and the Metropolitan Elevated Railway Company were leased to the Manhattan Railway Company (first organized in November, 1875), for 999 years, "in order, by means of one management, to avoid dangers of level crossings and also to perfect the system of Rapid Transit." This was known at the time as the Tripartite Agreement of May 20th, 1879.

The authorized rates of fares at that time were 10 cents for any distance of less than 5 miles, and not to exceed 15 cents for a through passage between the Battery and the Harlem River, except on "Commission Trains", which ran between 5.20 and 7.20 A. M. and between 5 and 7 P. M., when the corresponding rates were 5 and 7 cents. The 5-cent fare was introduced for all hours on all lines on November 1st, 1886. It is interesting to note that as early as January 20th, 1879, the collection of tickets by the conductors was abolished, the passengers on and after that date depositing their tickets in the ticket boxes provided at the exit gates, on leaving the trains. This was changed on June 21st, 1880, to the present method of depositing the tickets in canceling boxes at the entrance.

Another elevated railroad company, the Suburban Rapid Transit Company, was originally chartered in October, 1880, but did not commence construction until November, 1883, when the work on the foundations for the Harlem River Bridge at 129th Street and Second Avenue was started. The road was opened for traffic between 128th and 143d Streets in 1886, extended to 166th Street and Third Avenue in 1887, to 169th Street in 1888, and to 177th Street in 1891. The line from 177th Street to Fordham Road was opened in July, 1901, and from Fordham Road to Bronx Park in May, 1902.

On June 4th, 1891, the Manhattan Railway Company assumed control of the Suburban Rapid Transit Company, and from that date,

therefore, was in possession of all the elevated lines in Manhattan and The Bronx.

During 1899 to 1901, the whole system was equipped for electric traction, and since June 25th, 1903, all trains have been run by electric motive power.

On April 1st, 1903, the Manhattan Railway Company leased the system to the Interborough Rapid Transit Company, which since then has operated the lines.

General Description.

Scope of Work.—The "Manhattan Elevated Improvements" were authorized by a certificate, dated March 19th, 1913, issued by the Public Service Commission of the First District, State of New York, to the Interborough Rapid Transit Company, and formed part of a comprehensive scheme for rapid transit railroads in the City of New York. "Manhattan Elevated Improvements" involved in general the addition of a single continuous express track, with express stations, to the Second, Third, and Ninth Avenue Elevated Lines of the Manhattan Railway Company, operated by the Interborough Rapid Transit Company.

Before these improvements were completed, there existed in certain places on all the lines a center-track construction, and partial express service was in operation on the Third and Ninth Avenue lines; the "Manhattan Elevated Improvements" provided for a continuous express service during the rush hours, down town in the morning and up town in the evening. Two express tracks, of course, would have been better than one, but the cost of providing them would have been prohibitive, as it would have meant actually rebuilding completely the existing lines, and, in addition, it is doubtful if a permit would have been granted for a continuous four-track structure, on account of the objections of property owners. The single express track serves the purposes for which it was intended, namely, to carry the largest number of people speedily to and from their places of business, and to relieve congestion on the existing tracks. The congestion is never due to the operation of trains on the running track, but entirely to the stopping at stations; the more passengers there are to take or leave a train at a platform, the longer the train has to stop at the station; if the number of passengers reaches more than a certain limit, the length

of the stop increases at a much greater ratio than the number of passengers. The single express track relieves the condition of doubling the platform capacity for train service at the points, and at the time, of greatest congestion. The express trains, of course, return on the local tracks, but, as they return empty, in the direction of light traffic, no congestion is produced by them.

Track Changes at Junction Points and Terminals.—As just stated, the general purpose of the work was to provide a single continuous express track in addition to the existing local tracks. This work, however, involved generally at junction points and terminals, extensive track changes, which again necessitated considerable changes in the structure. This applies particularly to the junction points at Chatham Square, at 129th Street and Second Avenue, at 53d Street and Ninth Avenue, and to the terminals at City Hall and at 155th Street and Eighth Avenue.

At Chatham Square, the Second and Third Avenue Lines intersect. Before the reconstruction, the Second Avenue trains continued to South Ferry and the Third Avenue trains either to South Ferry or City Hall. There was a station just north of Chatham Square, in the Bowery, for all Third Avenue trains, and another at the east side of Chatham Square for all South Ferry trains. It was desired, as a part of the reconstruction work, to continue, in addition, the Second Avenue Line to City Hall. The express tracks, both on the Second and Third Avenue Lines, stop just north of Chatham Square, but, nevertheless, the new track layout contains eight tracks through Chatham Square, two of which were overhead in order to avoid the dangers and delays due to grade crossings. The new track layout in this vicinity is shown on Fig. 1. Commencing at City Hall Station, the Third Avenue tracks run on the lower deck past a new island platform at the west side of Chatham Square. Just north of this platform they divide into three tracks—one for the express trains—and continue north through the Bowery. The Second Avenue tracks start on the upper deck at City Hall Station, but come down to the lower grade when reaching Chatham Square, and run past a platform parallel to the Third Avenue City Hall platform into Division Street, where they divide into three tracks—one for the express trains. On the South Ferry Branch, the Second Avenue Line takes the old tracks past the old station at the east side of Chatham Square and connects with the Second Avenue

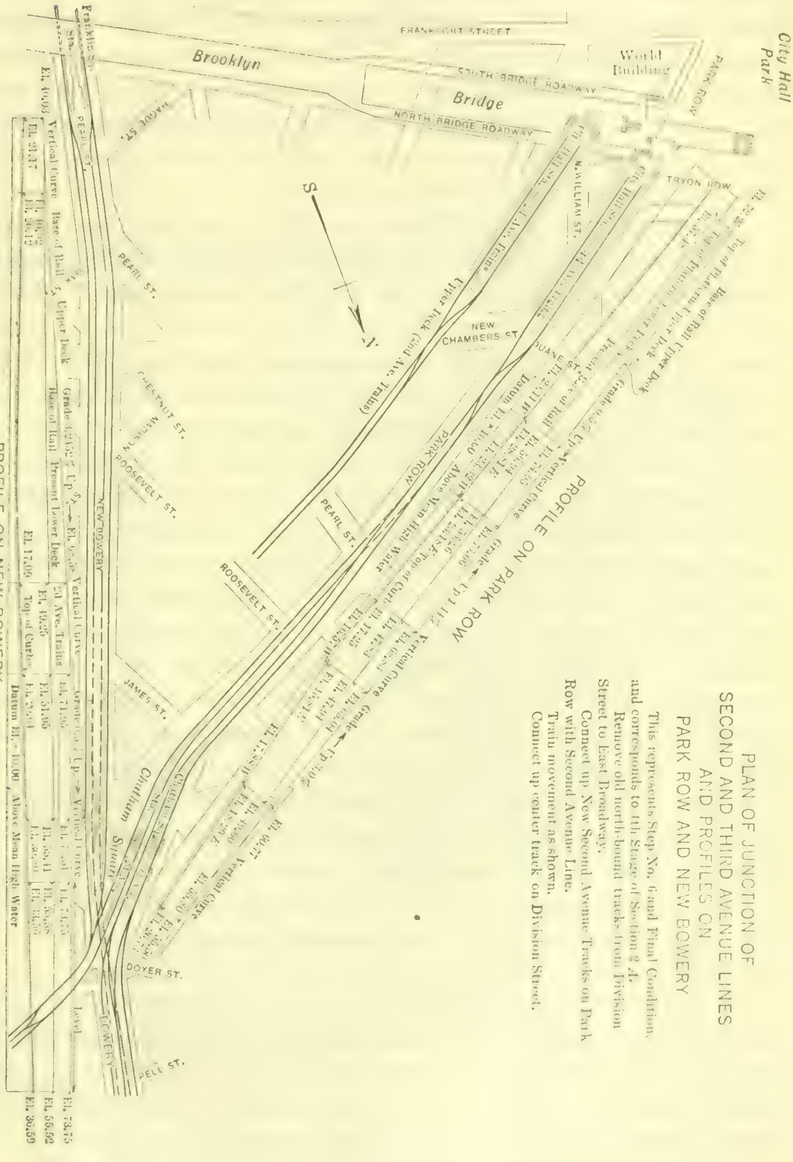

PLAN OF JUNCTION OF
SECOND AND THIRD AVENUE LINES
AND PROFILES ON
PARK ROW AND NEW BOWERY
This represents Step No. 6 and Final Condition,
and corresponds to 4th Stage of Section 2-4.
Remove old north-bound tracks from Division
Street to East Broadway.
Connect up New Second Avenue Tracks on Park
Row with Second Avenue Line.
Train movement as shown.
Connect up center track on Division Street.
PROFILE ON PARK ROW
PROFILE ON NEW BOWERY
City Hall Park
World Building
Brooklyn
Bridge
FRANKFORT STREET
SOUTH BRIDGE ROADWAY
NORTH BRIDGE ROADWAY
PARK ROW
TRYON ROW
N. WILLIAM ST.
NEW CHAMBERS ST.
DUANE ST.
PEARL ST.
HAGUE ST.
CHESTNUT ST.
MADISON ST.
ROOSEVELT ST.
JAMES ST.
NEW BOWERY
Chatham Square
DOYER ST.
BOWERY
PELL ST.
Upper Deck (2nd Ave. Trains)
Datum El. - 10.00 Above Mean High Water
EL. 36.59
EL. 56.52
EL. 73.75

FIG. 1.

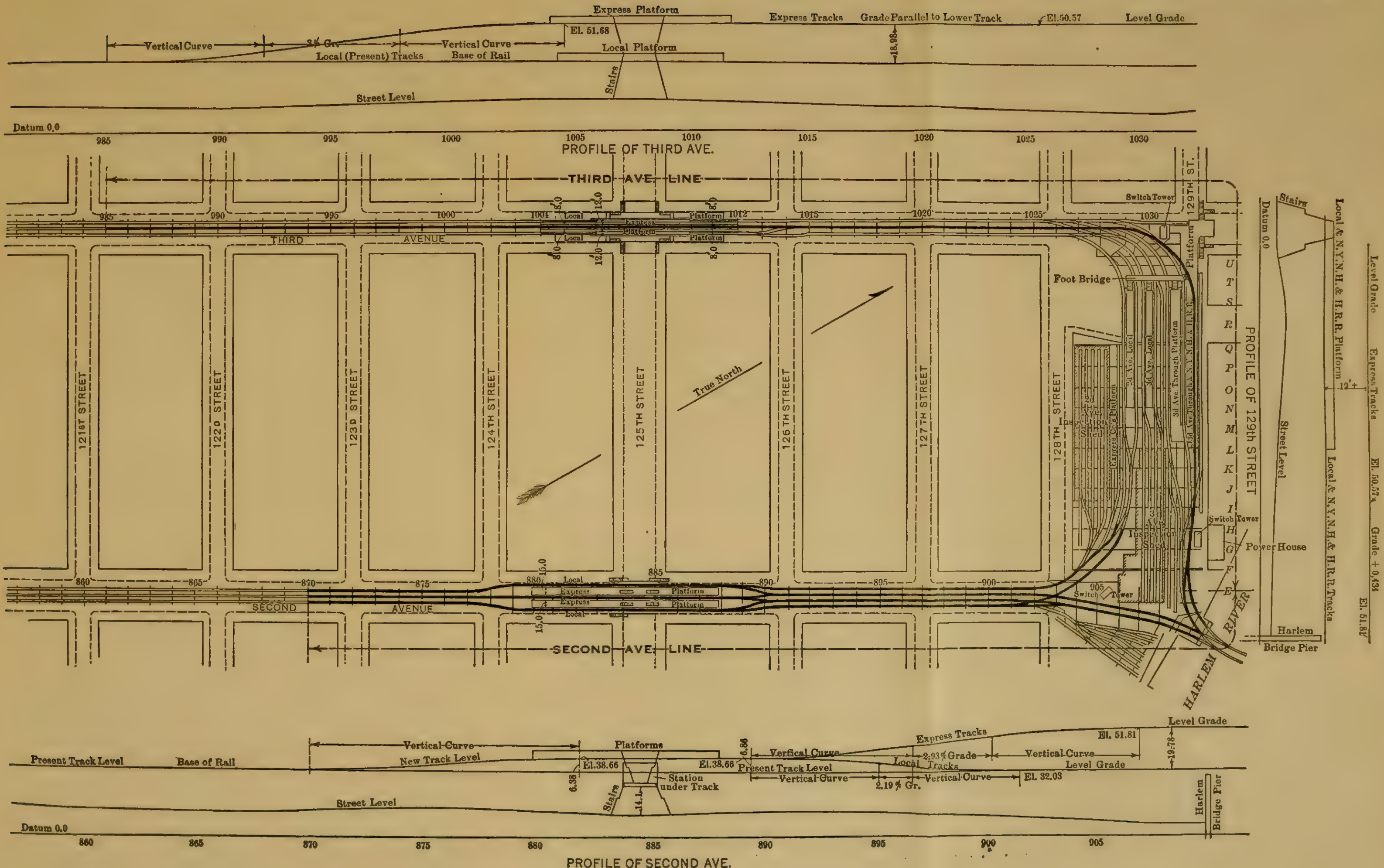
tracks from City Hall at Division Street. The Third Avenue tracks from South Ferry turn out from the old tracks just north of Franklin Square Station and, rising above the grade of the old tracks, run past a new platform above the platform of the old east station at Chatham Square, and, after crossing above the Second Avenue Line, run into the Bowery between the local and express tracks connecting to City Hall, and after descending to the level of these tracks, connect with them just south of Canal Street Station.

Another junction of the Second and Third Avenue Lines occurs at 129th Street and Second Avenue (see Plate XLIV). At 129th Street there is also a yard for the Second and Third Avenue trains. From north of 125th Street Station on the Third Avenue Line to 129th Street there existed, before the reconstruction, three tracks, the center one being the up-town main track and the east one a yard track. It was desired to retain these and to add an express track. The new express track, therefore, starts to rise south of 125th Street Station and continues above the center track to 129th Street and through 129th Street above the existing platform and tracks to a junction with the upper deck, Second Avenue tracks. In Second Avenue a three-track line continues to the north end of the new 125th Street Station. At this point the three tracks divide into four, the two outside continuing on the lower deck and connecting with the yard and with the local tracks of the Third Avenue Line. The two inside tracks rise above the grade of the lower deck and connect with the express track of the Third Avenue Line. This four-track line, at 129th Street and Second Avenue, then runs across the New Harlem River Bridge, with two of the tracks on the lower and two on the upper deck, and continues thus as a double-deck, four-track line to 145th Street, where a new two-track branch is being built to connect the two upper tracks with the West Farms Branch of the Subway. The two upper-deck tracks then converge into one, coming down to the grade of the local tracks at 147th Street.

At 53d Street and Ninth Avenue, the Sixth Avenue forms a junction with the Ninth Avenue Line. Formerly, the express track on the Ninth Avenue Line intersected the south-bound local track on the Sixth Avenue Line. This grade crossing was eliminated by raising the express track above the grade of the local tracks.

The terminal station at City Hall served formerly only the Third Avenue Line. The terminal facilities consisted of two tracks with a

PLATE XLIV,
PAPERS, AM. SOC. C. E.
DECEMBER, 1917.
GARDINER AND JOHANNESSON ON
ELEVATED RAILWAY IMPROVEMENTS.



center platform and two side platforms. These facilities were duplicated on an upper level directly above the existing terminal station, to serve the Second Avenue Line, and necessitated practically a complete rebuilding of the structure at this point.

The Ninth Avenue Line terminates at 159th Street, where there is a Company Yard. The terminal station was formerly at 155th Street, and this also formed the terminal of the Putnam Division of the New York Central Railroad. A connection of the Ninth Avenue Line with a new rapid transit line in Jerome Avenue is now being built, and, when it is completed, the terminal of the Putnam Division will be removed to the other side of the Harlem River, and the Elevated Railroad trains will run across the Putnam Bridge. The old station at 155th Street had two side platforms, with adjacent tracks and a center track. North of the station two of the tracks continued to the yard. The new arrangement cannot be completed until the Putnam Terminal is abandoned. There will be, when completed, two new island platforms between 155th and 157th Streets, each platform having two adjacent tracks, and in addition a fifth, which is a yard track, on the west side of the structure to the south end of the station platforms. At present the three westerly tracks and the west platform are completed and in operation. When all the work is completed, the west platform will be used as a terminal platform and the east platform for the through trains. In order to carry the north-bound trains to the terminal platform without a grade crossing, the express track is raised south of the station and the north-bound local track is divided into two branches, one of which runs under the express track to the west side of the structure. The express track is brought down to grade again immediately before reaching the station platforms.

Express Stations.—Although, in a number of cases, local conditions required express stations of special types, two standard types were used whenever the conditions permitted:

The type which required entire rebuilding of the existing station, has two island platforms, with the express track between the platforms, and a mezzanine station below the track structure, and was used when the head-room was sufficient to place the mezzanine station under the structure. The tracks, which have a standard spacing of 12 ft., are spread at the location of the station so as to make room for two plat-

forms. The west platform is used for down-town traffic only, the local trains discharging and receiving passengers on the west side of the platform and the express trains on the east side. The up-town express train receives and discharges passengers on the west side of the east platform and the up-town local trains on the east side. Stairs provide connection between the platforms and the mezzanine station, which occupies the space under the track structure between two adjacent bents, and contains the ticket office, waiting-rooms, toilet-rooms, and heater-rooms. Stairs connect the mezzanine station with the street.

The other standard type is the "hump" station, which was used when sufficient head-room could not be obtained to place a mezzanine station under the structure. This type of construction was conceived by George H. Pegram, President, Am. Soc. C. E., and provided a simple and efficient method of obtaining the necessary platform facilities for the express service. Before the reconstruction, such a station had two side platforms, one for up-town and one for down-town local traffic. In order to provide access to and from platforms for the express track, the grade of the center track was raised above that of the local tracks so that platforms to serve this track could be built above the local tracks. The standard car clearance required is 14 ft. 6 in. from the base of rail; the construction height of the express platform deck was made as shallow as possible, in order to obtain the least possible distance for the passengers to ascend. The necessary construction height was about 1 ft., and, as the standard height of the platform above the base of rail is 3 ft. 11 in., the grade of the express track was constructed about 11 ft. 6 in. above the local tracks. The grade of the express track was kept parallel to that of the local tracks, and the approaches were constructed with a maximum grade of about 3% and with vertical curves changing the grade 1% in 100 ft., corresponding to a curve of 10 000 ft. radius. Two stairways generally connect each platform with the corresponding local platforms. No additional station building rooms were required, as the old buildings contained all necessary facilities.

Work Divided into Sections.

For executive purposes, the work under "Manhattan Elevated Improvements" was divided into a number of sections as shown on Fig. 2.

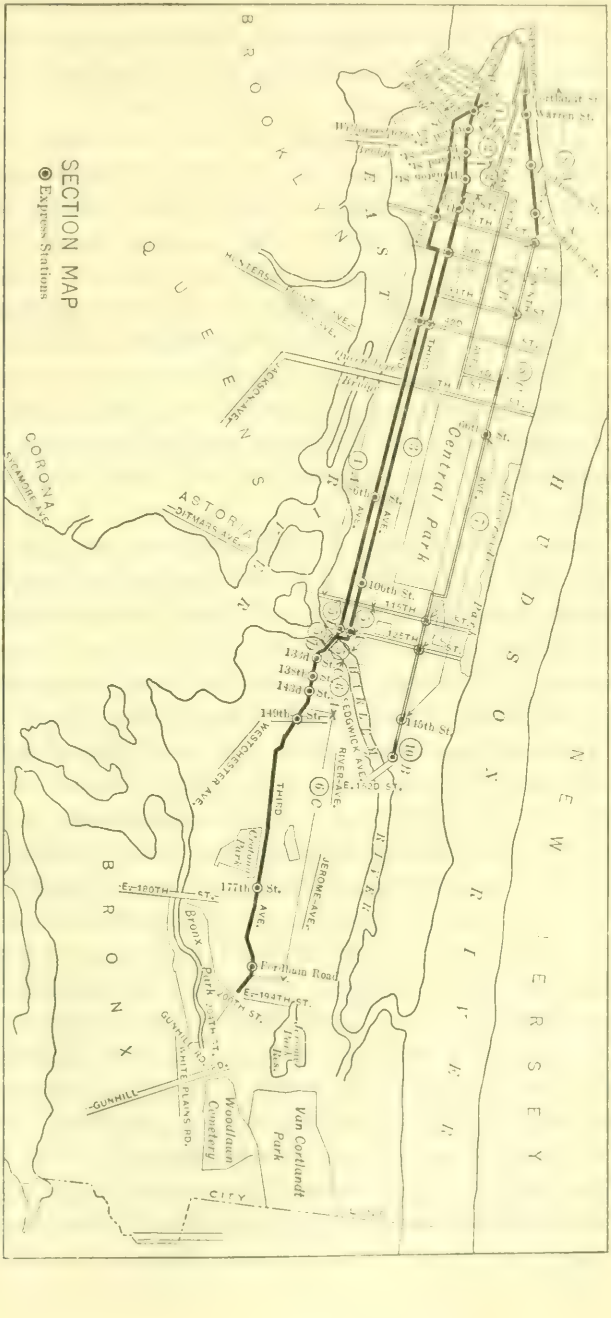

SECTION MAP
Express Stations
Central Park
HUDSON RIVER
NEW JERSEY
EAST RIVER
HARLEM RIVER
BROOKLYN
QUEENS
BRONX
ASTORIA
CORONA
Van Cortlandt Park
Woodlawn Cemetery
Bronx Park
Crotona Park
Fordham Road
177th St.
149th St.
145th St.
125TH ST.
116TH ST.
106th St.
THIRD AVE.
WESTCHESTER AVE.
JEROME AVE.
SEDGWICK AVE.
RIVER AVE.
WHITE PLAINS RD.
GUNHILL
DITMARS AVE.
JACKSON AVE.
SYCAMORE AVE.
Warren St.

FIG. 2.

As the type of structure and the class and condition of the work varied greatly, the description, for the sake of clearness, will be made for each section separately. The location of each section and the work involved is as follows:

Section No. 1.—This comprised the reconstruction of the Third Avenue Line on Park Row from the terminus at City Hall to Chatham Square, a total length of 1 960 ft., and on New Bowery from near Franklin Square to near Division Street, a total length of 1 840 ft.

The reconstruction in Park Row, including the City Hall Station, was practically a replacement of the old two-track structure with a new four-track structure, only some track stringers and platform girders of the old structure being retained. The old station, just north of Chatham Square, which served all Third Avenue trains, was removed, and a new station with two island platforms and four tracks was built in Chatham Square south of the old location to serve Third and Second Avenue trains to City Hall.

The work in New Bowery included the addition of a two-track structure, above the level of the present structure, supported on new cross-girders carried by the existing columns, and the reconstruction of the South Ferry Branch Station at Chatham Square to provide a train platform for the new Third Avenue Line upper-grade tracks, while the lower-level tracks served the Second Avenue Line.

The work also included the replacing of 52 existing column foundations with larger and deeper ones to carry the additional weight.

Section No. 2-A.—This comprised the reconstruction of the Third Avenue Line in the Bowery from Chatham Square to a point north of Delancey Street, a total length of 2 531 ft., and involved the replacement of the existing structure with a new five-track structure from Chatham Square to Canal Street and a three-track structure from Canal Street to Delancey Street. It included the building, at Canal and Grand Streets, of new express stations to take the place of old ones. Practically all the old structure was removed, the principal exception being the retention of a few of the old track stringers south of Canal Street. In addition to this work, 17 old column foundations on this section were replaced with new foundations.

Section No. 2-B.—This comprised the complete replacement of the old Third Avenue elevated structure in the Bowery from a point north of Delancey Street to Fifth Street, a total length of 2 728 ft. The old

structure consisted of two single-track lines, each supported on a single row of columns at the curb lines of the sidewalks. The new structure, which carries three tracks, is supported on a double row of columns in the roadway. The work in this section also involved the construction of a new express station to take the place of the old one at Houston Street.

Section No. 3.—This comprised the work on the Third Avenue Line in Third Avenue between 5th and 116th Streets. It included the construction of 5 104 lin. ft. of track structure to complete the center track, a portion of which was in existence prior to the commencement of this work, and involved placing new track stringers, replacing the existing cross-braces with new cross-girders, and reinforcing the column tops. The work also included the construction of new over-grade express stations at 9th, 23d, 42d, and 106th Streets, involving the replacement of 35 cross-girders and 70 columns under the stations with new and heavier material.

Section No. 4-A.—This comprised the completion of the center track on the Second Avenue Line from Chatham Square to 116th Street. Portions of the center track were built before the beginning of this work. The new work comprised the construction of 23 382 lin. ft. of new center-track structure, consisting of longitudinal track girders and bracing, and rebuilding the stations at 14th, 42d, and 86th Streets, to provide platforms for the express service, and the reconstruction of the 92d Street Station, which had a center platform, to provide room for the center track.

In this work is not included the reconstruction work between 50th and 60th Streets to connect to the Queensboro Bridge, as that does not form part of the "Manhattan Elevated Improvements."

Section No. 5-A.—This comprised the work to be done on the Third Avenue Line from 116th to 129th Street, and in 129th Street between Third and Second Avenues. The work involved the construction of an over-grade track commencing near 121st Street connecting with the upper-grade tracks of the new Harlem River Bridge near Second Avenue, and included the replacement of 24 cross-girders and 48 columns with new and heavier structures, the reconstruction of the stations at 125th and 129th Streets, and the replacement of 26 column foundations.

Section No. 5-B.—This comprised the work to be done on the Second Avenue Line in Second Avenue between 116th and 129th

Streets, and included raising the whole structure for a length of 1 387 ft., the maximum rise being 7.5 ft.; the construction of a new express station at 125th Street to replace the existing station at 127th Street, which was removed, and changing the three-track structure north of 126th Street to a four-track structure, with two of the tracks connecting with the upper deck of the new Harlem River Bridge and the other two tracks connecting with the Company's Yard at 128th Street and the lower deck of the new Harlem River Bridge.

Section No. 5-C.—This comprised the replacement of the Harlem River Bridge at 129th Street and Second Avenue, consisting of two approach spans, each about 102 ft. long, and a center swing span, 244 ft. long, carrying two tracks, with a new double-deck structure, carrying two tracks on each deck.

Section No. 5-D.—This comprised the addition of two upper-grade tracks to the structure from the north end of the Harlem River Bridge through the New York, New Haven and Hartford Railroad Company's Yard to 132d Street, and through the Company's Yard and property between 132d and 133d Streets. The new tracks were supported on new columns, which, in a number of cases, necessitated cutting the existing cross-girders of the lower-deck structure.

Section No. 6-A.—This comprised the work required on the Third Avenue Line from 133d Street, through the Company's private right of way, to 145th Street and through Third Avenue to 147th Street. The work included shifting the existing south-bound track sidewise for 13 ft. for a length of 2 800 ft.; constructing a new two-track overgrade structure between 133d and 143d Streets; reconstructing the stations at 133d, 138th and 143d Streets, and constructing an overgrade center track between 143d and 147th Streets; and involved the building of new piers and foundations.

Section No. 6-D.—This comprised the work on the Third Avenue Line between 147th Street and Fordham Road, involving the completion of the center track structure; replacing ten island-platform stations with new side-platform stations; reconstructing three stations as express stations; and replacing all the columns supporting the structure between 147th and 177th Streets with new and heavier columns.

Section No. 7.—This comprised the work on the Ninth Avenue Line required to reconstruct the stations at 66th, 116th, 125th, and 145th Streets to provide a continuous center track and express platforms.

Section No. 8-A.—This comprised the work on the Ninth Avenue Line required to construct a center track in Greenwich Street from Cortlandt to 9th Street, and center platforms at Cortlandt, Warren, Desbrosses, and Christopher Streets, and involved the construction of 10 540 lin. ft. of new track structure, consisting of track stringers and cross-girders; reinforcement of column tops; the replacement of 154 columns, and of 4 328 lin. ft. of old longitudinal girders with new girders for the existing south-bound track.

Section No. 8-B.—This comprised the reconstruction of the existing stations on the Ninth Avenue Line at 14th and 34th Streets to provide platforms for the express track.

Section No. 8-C.—This comprised the construction of over-grade crossing of the Ninth Avenue express track over the junction with the Sixth Avenue tracks at 53d Street and Ninth Avenue.

Section No. 10-B.—This comprised the reconstruction of the terminal station of the Ninth Avenue Line at 155th Street and Eighth Avenue, and involved the addition of an over-grade track between 150th and 155th Streets, the construction of two island platforms between 155th and 157th Streets, and the reconstruction of the track arrangement between 154th Street and the Company's Yard at 159th Street. A portion of this work cannot be completed until the structure on the east side of Eighth Avenue north of 155th Street, which is occupied by the Putnam Division of the New York Central Railroad, is vacated.

Design of Steel Structure.

The purpose of the work under the "Manhattan Elevated Improvements" was to improve the existing traffic facilities. The new track arrangement, therefore, was a matter of first consideration. When that was settled, the existing structure was surveyed for the purpose of determining its strength; the reinforcement needed, where new loads were added; the dimensions of the added structure, and the methods of connections between new and old work. The necessity of maintaining traffic during reconstruction was the governing feature in the design.

Specifications for Design.

The specifications for loads and unit stresses for the additions to the structure north of the Harlem River were different from those for the added structure in Manhattan, because the original structures

were built under different specifications, and it was desired to keep the stresses in the old and new structure uniform. The specifications for the new structure added to the elevated lines in Manhattan were as follows:

Dead Load.—The dead load shall consist of the estimated weight of the entire suspended structure, and is estimated to be 750 lb. per lin. ft. of track.

Live Load.—The live load for each track shall consist of a train of cars with 6-ft. wheel base, 33 ft. from center to center of trucks, and 15 ft. from center to center of adjacent cars, the cars having 20 000 lb. on each axle of the north truck and 15 000 lb. on each axle of the south truck.

The live load for canopy roofs shall be 30 lb. per sq. ft., and for station platforms, 80 lb. per sq. ft.

Lateral Load.—Transverse bents shall be designed for a force of 30 lb. per sq. ft. on the exposed surface of all trusses and the floor, as seen in elevation, and on the side of a train 10 ft. high, beginning 3 ft. above the base of rail.

Longitudinal Force.—Transverse bents and similar structures shall be designed for a longitudinal force of 10% of the live load applied to the rail.

Structures on curves shall be designed for the centrifugal force of the live load acting at a height of 5 ft. above the rail, proper account being taken of the super-elevation, to be determined by the Engineer for each case.

The centrifugal force is $f = \frac{W v^2}{g r}$.

in which $f =$ pounds,
$v =$ velocity, in feet per second,
$w =$ weight, in pounds, and
$r =$ radius, in feet.

Unit Stresses.—All structures designed for additional tracks or stations on existing elevated railroad structures in Manhattan shall be proportioned so that the maximum stress in axial tension on the net section shall not exceed 9 000 lb. per sq. in.

The axial compression on the gross section is $9\,000 - 40 \frac{l}{r}$,

in which l is the length of the member, in inches, and r is the least radius of gyration, in inches.

No column smaller than 12 in. shall be used.

Bending.—On extreme fibers of rolled shapes, built sections, and girders, net section, 9 000 lb. Roof work shall be designed for a fiber stress of 12 000 lb. per sq. in.

Shearing.—Rivets	....	7 500 lb.
Plate girder webs (gross section)	...	7 500 "
Bearing	....	15 000 "

For stiffeners, ground to bear, deduct area lost by grinding for fillet, in calculating effective area of stiffeners.

Bearing on Base Plates.—For direct loads, base plates of columns shall be proportioned for a pressure of 300 lb. per sq. in. on concrete (1:2½:5), adding 50% to this pressure for combined direct and bending stresses.

Bending.—On extreme fibers of rivets and pins, 15 000 lb.

Members subject to alternate strains of tension and compression shall be proportioned for the strain giving the largest section. If the alternate strains occur in succession during the passage of one train, as in stiff counters, each strain shall be increased by 50% of the smaller.

Whenever the live and dead-load strains are of opposite character, only 70% of the dead-load strain shall be considered as effective in counteracting the live-load strain.

Members subject to both axial and bending strains may be proportioned to a combined fiber strain 25% in excess of the allowed axial strain.

For strains produced by longitudinal and lateral or wind forces, combined with those from live and dead load and centrifugal forces, the unit strain may be increased 50% over the allowed axial strain. The section, however, shall not be less than that required if the longitudinal and lateral or wind forces are neglected.

Plate girders shall be proportioned either by the moment of inertia of their net section, or by assuming that the flanges are concentrated at their centers of gravity, in which case one-eighth of the gross section of the web, if properly spliced, may be used as the flange section.

The gross section of the compression flange of the plate girders shall not be less than the gross section of the tension flanges, nor shall

the strain per square inch in the compression flange of any beam or girder exceed $12\,000 - 200\,\frac{L}{B}$, in which L = the unsupported distance, and b = the width of the flange.

The flanges of plate girders shall be connected to the web with a sufficient number of rivets to transfer the total shear at any point in a distance equal to the effective depth of the girder at that point combined with any load that is applied directly to the flange. The wheel load, where the ties rest on the flanges, will be assumed to be distributed over three ties.

Trusses shall preferably have a depth of not less than one-tenth of the span. Plate girders and rolled beams, used as girders, shall preferably have a depth of not less than one-twelfth of the span, and if shallower trusses, girders, or beams are used, the section shall be increased so that the maximum deflection will not be greater than if the above limiting ratio had not been exceeded.

The specifications for the structure north of the Harlem River varied from the foregoing as follows:

Dead Load.—The dead load shall consist of the estimated weight of the entire suspended structure, and is estimated to be 750 lb. per lin. ft. of track.

Live Load.—The live load for each track shall consist of a train of cars, the cars being 46 ft. long, 32 ft. from center to center of trucks, 14 ft. from center to center of trucks of adjacent cars, 5 ft. from center to center of wheels of the same truck, having 25 000 lb. on each of the four axles.

The live load for canopy roofs shall be 30 lb. per sq. ft., and for station platforms, 80 lb. per sq. ft.

Unit Stresses.—All new structures shall be proportioned so that the maximum stress in axial tension on the net section shall not exceed:

In longitudinal girders, 10 000 lb. per sq. in.

In cross-girders, 12 500 " " " "

Axial compressions in columns............. $10\,000-40\,\frac{l}{r}$

" " " longitudinal girders ..$11\,000-40\,\frac{l}{r}$

" " " cross-girders..........$12\,500-40\,\frac{l}{r}$

in which L is the length of the member, and r is the least radius of gyration, in inches.

No column smaller than 12 in. shall be used.

Bending.—On extreme fibers of rolled shapes, built sections, and girders, net section:

Longitudinal girders .... 11 000 lb.

Cross-girders .... 12 500 lb.

Standards for Designs.—The standard overhead clearance is 14 ft. 0 in. above the top of rail, and the side clearance, 6 ft. 0 in. from center of track, except at platforms, where it is 4 ft. 7½ in.

The standard Manhattan car is 12 ft. 10½ in. high and 8 ft. 9½ in. wide. The track gauge is 4 ft. 8½ in.

Bending Moments in Statically Indeterminate Bents.—All cross-bents, except in special cases, where tower construction could be obtained were generally designed to resist moments both at the top and bottom of columns. In the course of the design, one, two and three-story bents were encountered, and for each type formulas for the moments at the top and bottom of columns were worked out. As the results may be of general interest, they are noted here for each type considered.

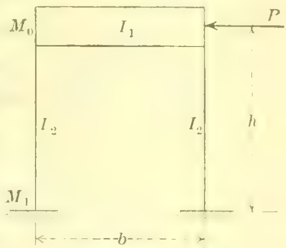


FIG. 3.

Type I.—Single-Story Bent; Both Columns Alike.

Let h = height of column, measured from bottom of column to center of cross-girder;

b = distance between centers of columns;

I_1 = moment of inertia of cross-girder;

I_2 = " " " " column;

P = horizontal force applied at center of cross-girder;

M_0 = moment at top of column;

M_1 = moment at bottom of column.

Then

$$M_0 = -\frac{3\,I_1\,P\,h^2}{2\,(b\,I_2 + 6\,I_1\,h)}$$

and

$$M_1 = +\frac{b\,I_2 + 3\,h\,I_1}{2\,(b\,I_2 + 6\,I_1\,h)}\,P\,h.$$

Type II.—Unsymmetrical One-Story Bent.

Let h = height of column, measured from bottom of column to center of cross-girder ;

b = distance between centers of columns ;

I_1 = moment of inertia of one column ;

I_2 = " " " " other column ;

I_3 = " " " " cross-girder ;

M_0 = moment at bottom of column I_1 ;

M_1 = " " top " " I_1 ;

M_2 = " " " " " I_2 ;

M_3 = " " bottom " " I_2 ;

P = horizontal force applied at center of cross-girder ;

and let

$$X_1 = P \frac{6\,h^2 \left(b + h\left(\frac{I_3}{I_1} + \frac{I_3}{I_2}\right)\right)}{b^2\left(\frac{I_1}{I_3} + \frac{I_2}{I_3}\right) + 4\,b\,h\left(\frac{I_1}{I_2} + \frac{I_2}{I_1} + 2\right) + 12\,h^2\left(\frac{I_3}{I_1} + \frac{I_3}{I_2}\right)b}$$

$$X_2 = \frac{b\,\frac{I_1}{2\,I_3} + h}{b\,\frac{I_1}{I_3} + h\left(\frac{I_1}{I_2} + 1\right)}$$

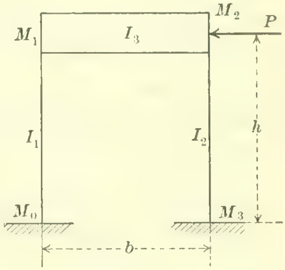


FIG. 4.

Then

$$M_0 = -X_1\,(b - X_2) + \frac{P\,I_1\,h}{I_1 + I_2}$$

$$M_1 = -X_1\,(b - X_2)$$

$$M_2 = X_1\,X_2$$

$$M_3 = X_1\,X_2 - \frac{P\,I_2\,h}{I_1 + I_2}$$

Type III.—Two-Story Bent; Both Stories Same Width; All Columns Alike.

Let h_1 = distance from center to center of cross-girders ;

h_2 = distance from center of lower girder to bottom of column ;

b = distance between centers of columns ;

I_1 = moment of inertia of upper cross-girders ;

I_2 = " " " " " column ;

I_3 = " " " " lower cross-girders ;

I_4 = " " " " " column ;

M_a = moment at top of upper column ;

M_b = moment at bottom of upper column ;

M_c = “ “ top “ lower “

M_d = “ “ bottom “ “ “

P_1 = horizontal force applied at center of upper cross-girder ;

P_2 = “ “ “ “ “ “ lower “

Further, let

$$A = \frac{P_1 \, h_1^2}{4 \, I_2}$$

$$B = \frac{P_1 \, h_2^2 + 2 \, P_1 \, h_1 \, h_2 + P_2 \, h_2^2}{4 \, I_4}$$

$$C = \frac{b^2}{12 \, I_1} + \frac{b \, h_1}{2 \, I_2} + \frac{b \, h_2}{2 \, I_4}$$

$$D = \frac{b \, h_2}{2 \, I_4}$$

$$E = \frac{b^2}{12 \, I_3} + \frac{b \, h_2}{2 \, I_4}$$

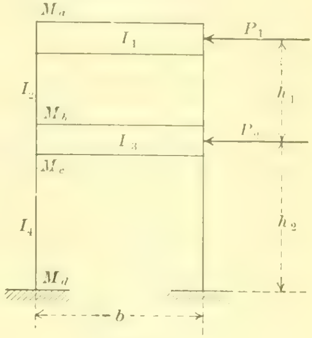


FIG. 5.

and

$$X_1 = \frac{(A + B) \, E - B \, D}{C \, E - D^2}$$

$$X_2 = \frac{B \, C - (A + B) \, D}{C \, E - D^2}$$

Then

$$M_a = - \frac{b}{2} X_1$$

$$M_b = - P_1 \, h_1 - \frac{b}{2} X_1$$

$$M_c = + P_1 \, h_1 - \frac{b}{2} (X_1 + X_2)$$

$$M_d = + P_1 \, (h_1 + h_2) + P_2 \, h_2 - \frac{b}{2} (X_1 + X_2)$$

Type IV.—Two-Story Bent; Upper Story Narrower than Lower Story; Symmetrical.

Let h_1 = distance from center to center of cross-girders;

h_2 = “ “ “ of lower cross-girder to bottom of column;

b = distance between centers of columns of upper bent;

c = “ “ “ “ “ “ lower “

d = “ from center of column of upper bent to center of column of lower bent;

I_1 = moment of inertia of upper cross-girder;
I_2 = " " " " " column;
I_3 = " " " " lower cross-girder;
I_4 = " " " " " column;
P_1 = horizontal force applied at center of upper cross-girder;
P_2 = " " " " " " lower "

Further, let

$$A = \frac{P_1\, b\, h_1^2}{8\, I_2} + \frac{P_1\, h_1\, (c^2 - b^2)}{2\, I_3}$$

$$B = \frac{P_1\, h_2^2 + 2\, P_1\, h_1\, h_2 + P_2\, h_2^2}{8\, I_4}\, c$$

$$C = \frac{b^3}{24\, I_1} + \frac{b^2\, h_1}{4\, I_2} + \frac{c^3 - b^3}{24\, I_3} + \frac{c^2\, h_2}{4\, I_4}$$

$$D = \frac{c^3 - b^3}{24\, I_3} + \frac{c^2\, h_2}{4\, I_4}$$

$$E = \frac{c^3}{24\, I_3} + \frac{c^2\, h_2}{4\, I_4}$$

$$F = \frac{P_1\, h_1\, (c^2 - b^2)}{16\, I_3}$$

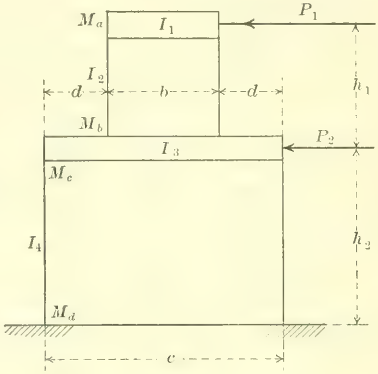


Fig. 6.

and let

$$X_1 = \frac{(A + B)\, E - (F + B)\, D}{C\, E - D^2}$$

$$X_2 = \frac{(F + B)\, C - (A + B)\, D}{C\, E - D^2}$$

Then

$$M_a = -\frac{b}{2} X_1$$

$$M_b = -\frac{b}{2} X_1 + \frac{P_1\, h_1}{2}$$

$$M_c = -\frac{c}{2} (X_1 + X_2) + \frac{P_1\, h_1}{2}$$

$$M_d = -\frac{c}{2} (X_1 + X_2) + \frac{P_1\, (h_1 + h_2)}{2} + \frac{P_2\, h_2}{2}$$

Type V.—Three-Story Pin-Connected Bent.

Let h_1 = height of upper story;
h_2 = " " middle "
h_3 = " " lower "
b = distance between centers of columns;

M_1 = moment at bottom of upper column;

M_2 = moment at bottom of middle column;

M_3 = moment at bottom of lower column;

P = horizontal force applied at top of bent;

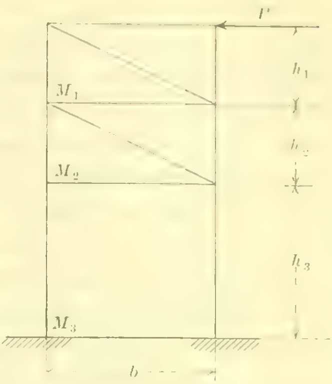


FIG. 7.

and let

$$p = \frac{h_2 h_3 (4 h_1 + 3 h_2) + 6 h_3^2 (h_1 + h_2)}{h_2 (4 h_1 + 3 h_4) + 12 h_3 (h_1 + h_2)}$$

$$q = h_3 - \frac{2 h_2 (h_1 + h_2)}{2 h_1 + 3 h_2}$$

$$X = P \left(1 - \frac{3 h_2 h_3^2}{h_1 (4 h_1 h_2 + 3 h_4^2 + 12 h_1 h_3 + 12 h_4 h_3)}\right)$$

Then

$$M_1 = -\frac{1}{2} (P - X) h_1$$

$$M_2 = -\frac{1}{2} P (h_3 - p)$$

$$M_3 = -\frac{1}{2} P p$$

General Details of Design.

New columns, supporting the structure in the street, were generally made 15 in. square, built up of two 15-in. channels with the flanges turned in, four angles and a web-plate. Using 50-lb. channels, 5 by 3½ by ¾-in angles, and 13 by ½-in. web, the section modulus of such a column is 294 in.^{3} in the direction parallel to the web and 129 in.^{3} in the direction square to the web. The columns were nearly always placed with the web in the same direction as the cross-girders, as the lateral bending stresses in the columns, due to wind and centrifugal forces, are greater than the longitudinal bending stresses, due to braking of trains. Columns on the sidewalk, supporting station structure only, were generally built up similar to the track structure columns, but 12 in. square.

Generally, the columns had to be designed so that they could be erected under the cross-girder, with the foundations—including the

anchor-bolts—in place. The columns, therefore, were made detachable, but capable of resisting the bending moments that might come on them. The original design had a lower part made up of plates and shapes, with holes for the anchor-bolts. The upper part of the base, which was riveted to the column, was made narrow enough to pass between the anchor-bolts. The upper and lower sections were to be connected with flat plates, riveted in the field. In the design of a similar type the upper and lower parts are connected with bolts.

The first design was not used at all, and the second design only in a few cases, as the bridge shops requested that the design be changed, on account of difficulty in fabricating the lower sections. Therefore, another type of detachable base, shown by Fig. 8, was designed. The lower part of this base consisted of a rolled steel slab, 6 in. thick, with four holes to fit over the anchor-bolts in the foundation. The upper part of the base, which was narrow enough to pass between the anchor-bolts, was riveted to the column shaft, and was connected to the slab with four screw-bolts, 2 in. in diameter, fitting into tapped holes in the slab.

A stiff connection between the column and the cross-girder was obtained, when the column was at the end of the cross-girder, by running the column shaft to the top of the cross-girder and riveting the end of the girder to the column. When the column was under the cross-girder, the stiff connection was obtained by riveting four angles to the column, extending above the top of the column; these angles were connected to the bearing stiffeners of the cross-girder with plates riveted to both angles and stiffeners. The column and the cross-girder were further connected with plate brackets riveted to the sides of the column and the bottom of the cross-girder.

The cross-girders were generally plate girders in preference to lattice girders, due to the simplicity of design and also the facility with which a load can be placed at any point on a plate girder, without disturbing the uniformity of the design. This is specially advantageous for a structure of the kind herein described, as the location of the loads, due to track stringers, platform girders, overhead structures, etc., may vary from bent to bent.

The track stringers also were usually plate girders, but, where it was desired, either to keep the construction as open as possible, or to conform to the existing design, lattice stringers were used. Whenever

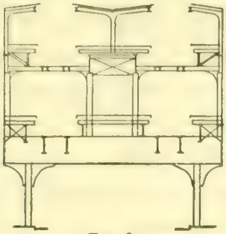


Fig. 8

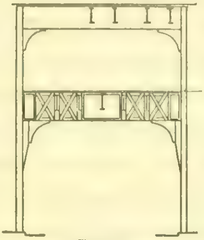


Fig. 9

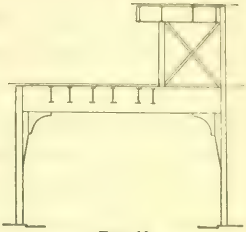


Fig. 10

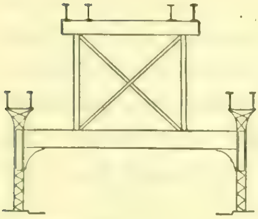


Fig. 11

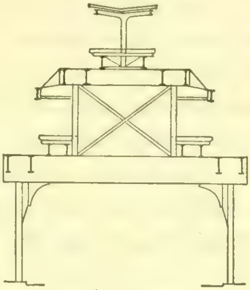


Fig. 12

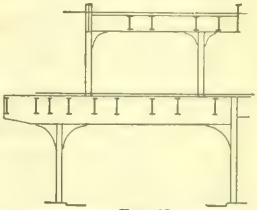


Fig. 13

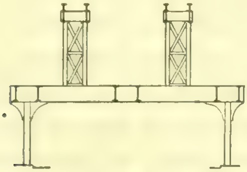


Fig. 14

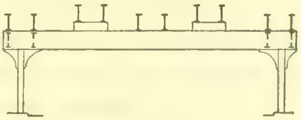


Fig. 15

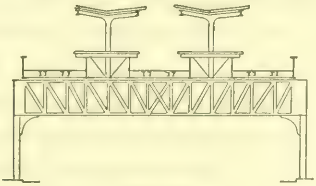


Fig. 16

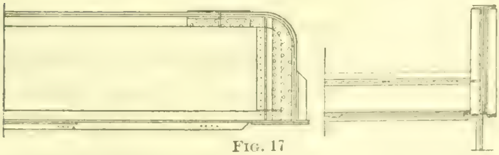


Fig. 17

the conditions permitted, the track stringers were designed with end lips, which supported the stringers, either on top of the cross-girders or on seats attached to the web of the cross-girders. The lip connection makes the girder simple to erect, is stiff, and occupies little construction height. Experience has shown that the rivets in the connection of framed-in track girders frequently break.

Expansion was provided for by slotting the holes at the connection and letting the girders slide on the seat. Generally, expansion joints were provided at one end of every second track stringer.

Details of Design.—Section No. 1.—A portion of the work included under Section No. 1 was the reconstruction of the elevated line in Park Row between City Hall and Chatham Square. The old two-track structure in Park Row carried the two-track Third Avenue Line to the terminal at City Hall. The reconstruction involved the addition of two tracks to the structure in Park Row for the Second Avenue Line; a terminal for the Second Avenue Line at City Hall; and a new station at Chatham Square capable of serving all four tracks of the new structure.

The old station at City Hall was reached from the one-story approach to the Brooklyn Bridge; it consisted of two tracks, a center platform on steel supports, and two side platforms altogether of wooden construction. These facilities were retained, but improved in details for the Third Avenue Line, and a new Second Avenue Line Station with similar platform and track facilities was built immediately above the Third Avenue Line Station, thus making a double-deck station. There was no room to carry all four tracks at one level, because Park Row is too narrow there for such construction. Fig. 8 shows a cross-section of the new double-deck station. On account of the added load, it was necessary to replace the old cross-girders and street columns with new and heavier ones. The upper-deck structure was supported on four rows of columns resting on top of the new lower-deck cross-girders. As it was desired to keep the level of the upper-deck platforms as low as possible at the point of access at the south end of the stations, the upper deck was designed as a through bridge construction, with longitudinal girders supported directly on the four rows of columns. The lower deck was level; the upper deck was laid on a grade of $\frac{1}{2}\%$ rising from the south to the north end of the station. This was done in order to gain sufficient clearance at the north end for

the standard cross-girder construction where it was required, on account of the cross-overs from in-bound to out-bound tracks. The upper-track floor construction consisted of shallow cross-girders or beams riveted to the through girders and track stringers framed into these cross-beams. In order to make the floor as shallow as possible, riveted box girders were used at the south end of the station. As the construction height increased, 15 and 18-in. Bethlehem girder beams were used instead. The track stringers consisted of two I-beams under each rail, in order to get sufficient bearing surface for the ties. The center platform was supported on top of the longitudinal girders, and the outside platforms partly on the longitudinal girders and partly on the track floor. The columns supporting the upper-deck structure were 12 in. square, and consisted of two 12-in. channels with a diaphragm of four angles and a web-plate. In order to obtain a fixed connection at the bottom, the columns were connected to the stiffeners of the supporting cross-girders with tension plates, and to obtain a fixed condition at the top of upper-deck columns, the diaphragm construction of these columns was continued up to the top of the through girders and the latter were riveted to these diaphragms. At expansion joints, however, the free ends of the girders rested on seats on the columns.

Between the north end of the City Hall Station and the south end of the new Chatham Square Station, the two upper-deck tracks were brought down to the level of the old structure, in order to connect with the tracks of the Second Avenue Line at Chatham Square. This was accomplished by running the lower-deck tracks to the west side of the structure through Park Row, and the upper-deck tracks to the east side. The first few bents of the upper deck north of City Hall Station had to span the entire width of the structure, as shown by Fig. 9, but as soon as the upper and lower-level tracks were sufficiently clear of each other, the upper deck was supported on two-column braced bents supported on the lower-deck cross-girders east of the lower-deck tracks, as shown by Fig. 10. These upper-deck bents were also braced longitudinally in pairs, so as to form a tower construction. The lower-deck columns had to be renewed on account of the added load, and where they coincided in position with the upper-deck columns they were built in one piece from the street to the upper deck. For appearance sake, it was desired to avoid transverse bracing below the lower

deck; as, however, the regular column section below the lower deck was not sufficient to carry the combined direct and transverse bending stresses without bracing, and as it was not permitted to use a width of column greater than 15 in., where it might interfere with the use of the sidewalk, the necessary additional strength was obtained by widening the column from a point 12 ft. above the sidewalk. This was accomplished by using two angles on the street side of the columns instead of the customary 15-in. channel. At a distance of 12 ft. above the sidewalk, the web was spliced and its upper part extended through the angles and reinforced with flange angles. Some of the existing lower-deck lattice cross-girders were retained, where there was no additional loading, but, on account of the track girders being changed in position, the latticing of cross-girders was removed at such points, and a web-plate was riveted to the cross-girder instead, as shown by Fig. 9.

The remainder of the work under Section No. 1 comprised the construction of an upper-grade station above the existing station on the east side of Chatham Square and a two-track approach to this station both at the north and south ends. The old lower-deck station was formerly used for both Second and Third Avenue Lines on the South Ferry Branch. After the reconstruction the lower deck was used for Second Avenue Line trains only, and the new upper deck for Third Avenue Line trains. The approach to the upper station commenced in the New Bowery just north of Franklin Square Station. The structure involved in the New Bowery consisted, before the reconstruction, of two single rows of columns on the sidewalks, each row carrying a single-track structure. The old columns opposite were braced with stiffening struts across the street. The columns were of the "fan-top" type, that is to say, they were made of two channels latticed together, and the channels were flared at the top, so that a track stringer came directly on top of each channel.

The new tracks were carried on cross-girders, which, at the beginning of the ramp at the south end, were set on the top of the old columns between the old track stringers, because the head-room above the street was small. These cross-girders were made in three pieces, in order to façilitate the erection. Farther north, where the head-room was greater, the new cross-girders were framed into the sides of the column shaft, as shown by Fig. 11; this method was preferred,

because the new structure could be erected without shoring and without interfering with traffic. In order to transfer the additional loading uniformly to the column, a diaphragm was riveted inside of the channels of the column, thus carrying the cross-girder reaction equally into both sides of the column. The upper tracks were supported by columns centered on the tracks and resting on the new cross-girders. The two columns of each bent were braced together, and every second pair of bents was also braced together, so that the upper structure formed a completely braced tower construction, resting on top of the new cross-girders. Additional longitudinal bracing was also provided between the old columns, so as to decrease the unsupported length of these columns and thus add to their carrying capacity.

Fig. 12 shows a cross-section of the structure at the Chatham Square Station. On account of the additional loading, it was necessary to replace the existing columns and cross-girders with new and heavier ones. This figure shows a cross-section at the south end of the station, where the lower center platform is divided into two platforms with an open space between. At the crossing with the City Hall branch tracks, the track layout necessitated a span length of about 75 ft. A through bridge construction, as shown by Fig. 13, was used in order to obtain sufficient depth for the stringers without encroaching on the head-room. The location of the columns was determined by the track arrangement on the lower deck.

Details of Design.—Section No. 2-A.—The work under Section No. 2-A comprised the reconstruction of the Third Avenue Line in the Bowery between Chatham Square and a point north of Delancey Street. Prior to the reconstruction, the old Chatham Square Station of the Third Avenue Line was in the Bowery just north of Chatham Square. The old station had a wide island platform serving the existing tracks. The northerly part of the platform was divided through the center, and the space was occupied by two pocket tracks, with an island platform between them, raised slightly above the grade of the main structure. The pocket tracks ended just south of the old Canal Street Station, where they turned into the main tracks. North of this point, the line was a two-track structure. The old Canal Street Station had two side platforms extending to the south of Canal Street. North of Canal Street, the structure consisted of two rows of fan-top columns support-

ing the track structure. The old Grand Street Station was between the tracks, with the platforms extending to the south of Grand Street.

The new track layout south of Canal Street retained the old main tracks in approximately their old location for local service on the City Hall branch. A center track was added on the same level for express service on the City Hall branch, and between the center track and the local track were added two over-grade tracks to connect with the South Ferry branch. The old pocket tracks and the station north of Chatham Square were removed. Fig. 14 shows a cross-section of the new structure just north of Chatham Square. On account of the additional loading, the existing columns and cross-girders had to be replaced with new and heavier ones. The existing track level, which it was necessary to maintain at the Chatham Square Junction, was close to the street, so that the new cross-girders had to be made very shallow compared with their span, in order to obtain the required 14 ft. of head-room under the structure. The use of twin girders was first considered, but as a plate-girder construction was practically necessary, this type of girder was abandoned on account of the difficulties in making connections to and maintaining a box-girder construction. Therefore, single plate girders, with heavy flanges, 20 in. wide, were used. On account of the width of the flanges, it was necessary to place track ties on top of the cross-girder, in order not to space the ties too far apart. The tops of the cross-girders, therefore, were made flush with the tops of the track stringers, and the latter were supported on seats attached to the web of the cross-girders and had the same depth.

The upper-level tracks were supported separately by framed two-post bents of the same width as the distance between the supported track stringers. The bents on two adjacent cross-girders were braced together longitudinally, and, in order to prevent sidewise overturning, the bents were spliced to the supporting cross-girders. The upper local track stringers were supported on the bents with lips 18 in. deep.

In order to facilitate the construction, the old locations of the columns on the sidewalk were retained; this, however, involved skew connections between the columns and the cross-girders, as well as between the cross-girders and the track stringers.

At the south end of the section, where the existing track grade was maintained, the old track stringers of the existing main tracks were used in the new work, after the stringer ends had been remodeled to

match the new cross-girders. Farther north, where the new track grade was higher than the old one, new track stringers were used, because the maintenance of traffic necessitated first, the erection of the new cross-girders and the framing of the track stringers at the existing level, and, later, the raising of the track stringers to the new level. To avoid this double reconstruction of the old track stringers, which would have been unsatisfactory and expensive, they were remodeled for the temporary connection to the new cross-girders, and new stringers were provided for the final position. Fig. 15 shows the temporary and permanent arrangement at the connection of the stringers and the cross-girder. The old stringers were carried temporarily on seats supported by the column brackets. When the old stringers were removed, additional stiffeners were riveted to the cross-girder at points where the new stringers were supported on the top of the cross-girder. Fig. 15 also shows the seats supporting the upper deck, at a point where the upper grade approaches the elevation of the lower deck.

The new express station at Canal Street was built as a mezzanine station. In order to obtain sufficient head-room for the mezzanine structure, the new track grade was raised above the old grade. After the design was completed, the City authorities decided to raise the grade of Canal Street in conjunction with the approach to the Manhattan Bridge. This necessitated a further raising of the new track level, which was obtained by a shortening of the vertical curves.

Fig. 16 shows the cross-bents supporting the mezzanine station. Whenever it was possible, the columns supporting the structure were placed in the roadway. The cross-girders were 8 ft. deep, and the mezzanine station occupied the full depth of the girders, the bottom of the mezzanine floor being nearly flush with the bottom of the girders. The longitudinal girders in this span were set on top of the cross-girders and carried the mezzanine station, the platforms, and a through bridge construction for supporting the track. Fig. 17 shows the details of the longitudinal girders. They were plate girders, and the web was extended below the bottom flange of the girder in order to provide means for connecting the hangers supporting the mezzanine structure.

A cross-section of the remainder of the structure of the Canal Street Station is shown by Fig. 18. The track stringers are carried on seats attached to the web of the cross-girder, and the top of the stringers is 3 in. below the back of the top flange angles of the cross-girders. This

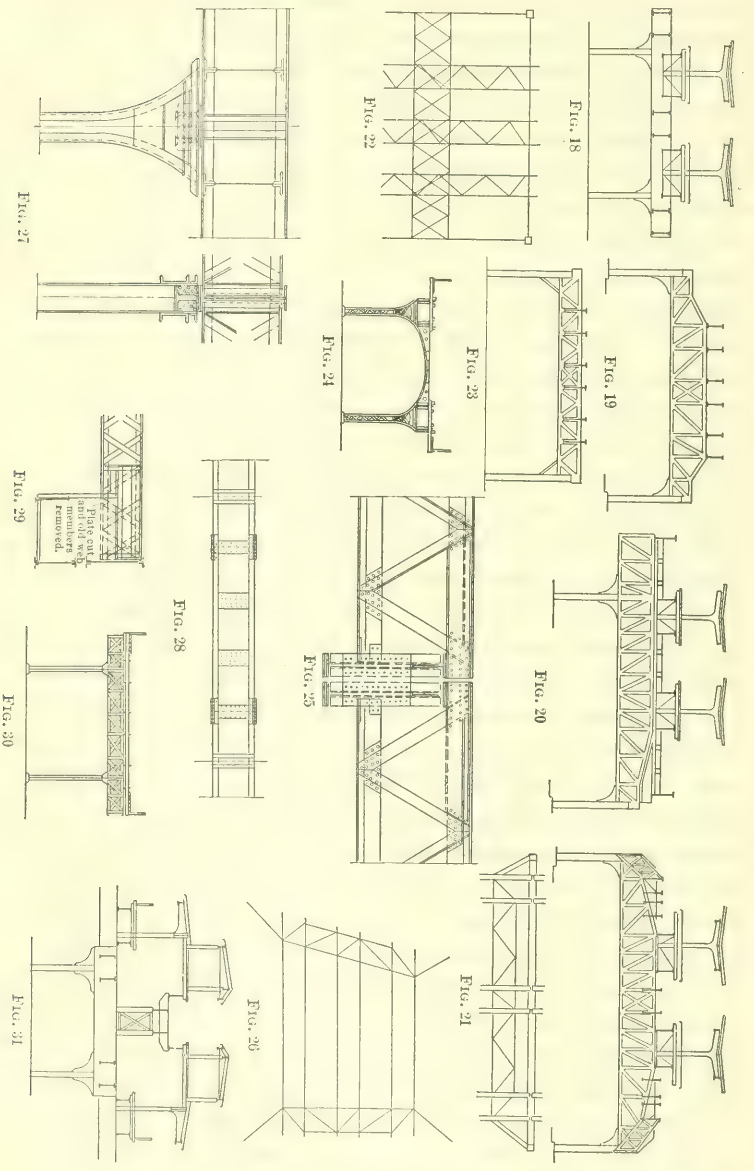


FIG. 18
FIG. 19
FIG. 20
FIG. 21
FIG. 22
FIG. 23
FIG. 24
FIG. 25
FIG. 26
FIG. 27
FIG. 28
FIG. 29
Plate cut and old web members removed.
FIG. 30
FIG. 31

arrangement permits the rail to be carried clear of the cross-girder without undue spacing of the ties. The bottom flange of the stringers was made flush with the bottom of the cross-girders, and the bottom flanges of the stringers and cross-girder were connected with a plate. The platform stringers were 4 ft. deep, and rested, with a lip 2 ft. 4 in. deep, on the top of the cross-girders.

The structure was braced with diagonal struts from the center track stringers to the columns, in order to prevent horizontal deflection of the cross-girders due to braking.

From north of the Canal Street Station to the south end of the Grand Street Station, the distance between street-car tracks and curb necessitated placing the columns on the sidewalk. The existing structure consisted of two rows of columns supporting the track structure, one on each side of the street. The new grade of the tracks was considerably higher than the old one on account of new track due to the mezzanine stations, but it was necessary to keep the ends of the cross-girders below the existing track work, as traffic on the existing tracks had to be maintained during the erection of the new structure. The cross-girders, therefore, were designed as shown by Fig. 19. The bottom flange of the cross-girders was kept straight; the middle portion of the top flange was kept high, partly to get sufficient depth of the girder and partly to bring the track stringers, which were set on top of the cross-girder, to the proper level. At the ends, the top flange of the cross-girder was brought down to such a level that the girder could be erected without interfering with the ties of the existing tracks.

The construction through the Grand Street Station was similar to that just described except that twin girder construction was used to carry the additional load. Fig. 20 shows one of the bents carrying the mezzanine station at Grand Street. As one of the columns could be placed in the street, it was not necessary to drop the one end of the girder, because the cantilever end was not erected until the traffic on the existing track was abandoned. The seat at the other end of the cross-girder was added after traffic had stopped on the existing track, and was required for the support of the through bridge girder carrying the track floor. The construction of the mezzanine station was the same as that at Canal Street. Fig. 21 shows the details of the bents carrying the platforms outside of the mezzanine station. The seats were made in two pieces, because, on account of the necessity

of maintaining traffic, the platform could not be constructed to its full width at once. In order to prevent longitudinal deflection of the cross-girders, horizontal trusses were provided; they were riveted to the bottom flanges of the stringers, and braced to the columns, as shown by Fig. 21.

From north of Grand Street Station to a point near Delancey Street, bents of the same type were continued, owing to the necessity of keeping the columns on the sidewalks. As the distance from the stringers to the columns was too great to use a diagonal brace, the method of bracing was changed to the type shown by Fig. 22. A horizontal stiffening truss, placed at the center of the span, connected the bottom flanges of all the stringers and extended at each end out to, and was connected with, the top flanges of vertical stiffening trusses between the columns.

Details of Design.—Section No. 2-B.—In this section, which extended from near Delancey Street to near 5th Street, all the columns of the new structure, with the exception of a few near 5th Street, were in the roadway; columns of the old structure were at the curb line, and the whole new structure, therefore, could be erected without interfering with the old one, except at the connection at 5th Street. Fig. 23 is a typical section of the structure. As there were four street-car tracks, the columns of the bent were spaced 45 ft. from center to center. The columns were originally designed with fixed bases, capable of resisting the moments. It developed, however, during the construction, that it would be necessary to let a gas main pass directly through the center of the foundations on both sides of the street, and it was decided, therefore, that it would be safer to place a heavy steel slab on the top of the foundations. The detachable type of base, with steel slab 6 in. thick, therefore, was decided on, and, for the sake of uniformity, this base was used throughout on both sides of the street. The cross-girders were designed as twin lattice girders. The twin girder construction was used, because it was desired not to carry the structure higher than necessary. The cross-girders rested with lips on top of the columns, and were braced to the column shaft with angle brackets. The standard spacing of the track stringers was 5 ft. between the stringers of each pair and 12 ft. from center to center of tracks. The panel spacing was arranged so as to bring the panel points directly under the track stringers.

Fig. 25 shows the details of the ends of the track stringers. The girders were supported on the top of the cross-girder with lips. As it was impossible to avoid secondary stresses at the support, the top chord was reinforced with 12-in. channels. Fig. 26 shows the lateral stiffening trusses and the braces to the columns. It will be noted that the braces have the additional function of fixing the tops of the columns, the braces and the stringer to which they are attached acting together as a stiffening truss.

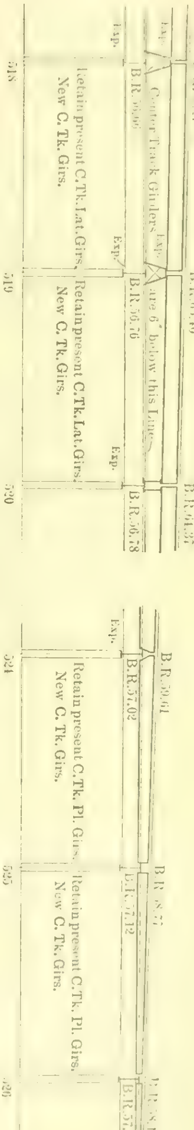
The Houston Street Station is included in this section; its general construction is similar to that of other mezzanine stations on the Bowery. As at the other stations, the depth of the cross-girders was made to conform to the height of the mezzanine. Through bridge girders carried the platform and track construction over the mezzanine station.

Details of Design.—Section No. 3.—Prior to the reconstruction there existed on the Third Avenue Line between 5th and 116th Streets, which portion comprised Section No. 3, a partial center-track construction, in fact, an express service was in operation north of 42d Street. South of the 42d Street Station center-track girders were in existence at a few isolated points, but, generally, the structure consisted of two tracks supported directly on top of the columns and braced together with an arch-shaped strut, as shown by Fig. 24. The columns were of the fan-top type, hereinbefore described. Originally, this structure was designed to carry only the load of the track supported directly by the columns, but, as a center track was added in places, in the course of time, it became necessary to reinforce the flaring portion of the column, and a design for the reinforcement was developed. This design, Fig. 27, was used also for the reinforcement of the columns included in the new work. The general principle of the design was that the load of the stringers supporting the outside tracks should be transferred to the columns as before, and that the load of the center track should be carried by a cross-girder to the center of the column, where it was supported on two I-beams, framed into two channels, riveted to plates, which were riveted to the flaring flanges of the column channels. These plates also strengthened the curved portions of the column channels which took the load of the outside track stringers. This somewhat involved construction was necessary, in order to be able to do the riveting in the closed space between the

channels and the plates. The new cross-girders were plate girders. To facilitate erection, they were made in three pieces, as shown by Fig. 28. The outside track stringers rested on the bottom flange of the cross-girders, and were supported directly by the column. The center track stringers were framed into the cross-girder at the fixed connections, and rested on seats at the expansion end. The new track stringers were plate girders.

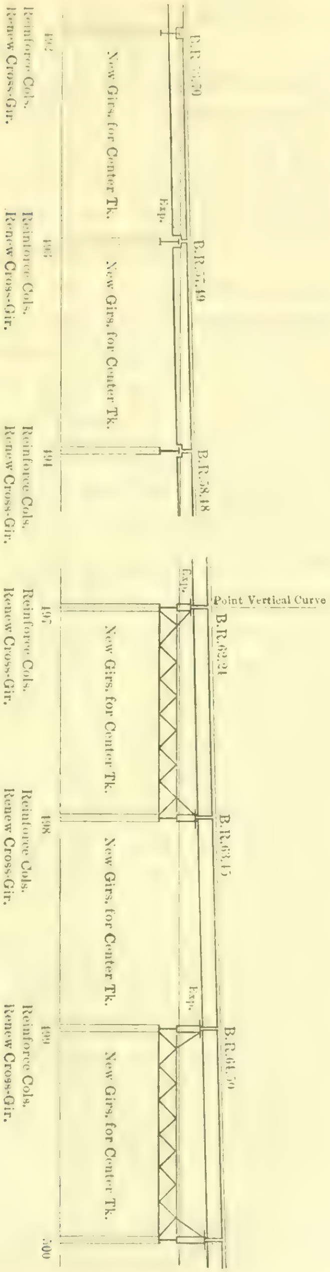
On this section, the stations at 9th, 23d, 42d, and 106th Streets were rebuilt for express service. As there was not sufficient head-room at any of these stations for a mezzanine station, the "hump" type of express stations was used. Fig. 32 shows portions of the southerly approach to the 42d Street Station. As there were no center track girders at this location prior to the reconstruction, new cross-girders and stringers were provided. The cross-girders had to conform to the elevation of the existing local tracks, while the center-track stringers were rising above the grade of the local tracks. In the first few spans, where the stringers came slightly above the top flange of the cross-girder, they were cut to make room for the cross-girder flange. When raised sufficiently, a lip construction was used until the stringer was clear of the cross-girder. Seats and bents were used thereafter to support the stringers. The bents consisted of two posts, placed 5 ft. from center to center, braced together, and set on top of the cross-girders. The bent was braced longitudinally with two diagonal struts supported on longitudinal lattice stiffening trusses framed into the cross-girders in the fixed spans.

The north approach differs somewhat in certain details, because the cross-girders and center-track stringers were in place prior to this work. The center-track stringers were retained in place, in order to stiffen the structure, and the raised track was constructed on top of these stringers. At the low portion of the approach, tapered girders, as shown by Fig. 33, were riveted to the top flange of the existing stringers. As the height of the new track above the old one became greater, plate-girder seats, resting on top of the track stringers close to the cross-girder, were used, and finally A-shaped towers, with the posts resting on the stringers. The reason for supporting the seats and the towers on the stringer ends, rather than directly on the cross-girder, was that the presence of the old track stringers, framed into



HUMP STRUCTURE NORTH OF 42d STREET, THIRD AVENUE LINE

Fig. 32.



HUMP STRUCTURE SOUTH OF 42d STREET, THIRD AVENUE LINE

Fig. 33.

the cross-girders, did not permit a satisfactory connection of the supporting bent to the cross-girder.

Fig. 31 shows a cross-section of the structure at the platforms. The ends of the express-track stringers which rest on top of the bent, are cut so as to form seats for two 24-in. I-beams which are flush at the top with the top of the stringers, and cantilever at each end, the cantilever ends supporting the inside girders which carry the express platform. The outside edge of the platform was supported on girders resting on columns which were again supported by the lower-level cross-girders. In order to make the vertical distance from the lower deck to the upper-deck platform as small as possible, the express platform construction was made very shallow. On account of the additional loading, due to the express platforms, the lower-deck cross-girders and columns were renewed.

At 42d Street there is a branch leading to the New York Central Terminal Station, to which passengers transfer from the Third Avenue Line Station at 42d Street. Before the reconstruction, access to this branch from the up-town platform at 42d Street was had by a bridge crossing over the tracks to the down-town platform, which connected directly with the branch platform. The overhead bridge interfered with the new overhead express station structure and therefore was removed, and in its place a bridge was provided under the lower deck structure. On account of the small head-room between the street and the structure, the depth of the stringers, under which the bridge had to pass, was cut down, as shown by Fig. 29. The old stringers were lattice girders. The remodeled portions of the stringers were reinforced with web-plates.

Details of Design.—Section No. 4-A.—Section No. 4-A comprised the Second Avenue Line from Chatham Square to 116th Street. Prior to this reconstruction work, a center-track construction existed on certain portions of the line. The work comprised the completion of the center-track structure; the double latticing of the existing center-track stringers; the reconstruction of the stations at 14th, 42d, and 86th Streets for express service; and the rebuilding of the island station at 92d Street.

Fig. 30 shows a cross-section of the structure before the reconstruction. The columns are of the six-section Phœnix type. The cross-

girders are twin lattice girders and the track stringers are lattice girders supported on top of the cross-girders by a lip 7 in. deep.

Where the work consisted of adding center-track girders only, no changes were made to the existing structure. As it was generally desirable to have the center track at the same elevation as the outside tracks, and necessary at switches and cross-overs, the new center-track stringers were also designed with a 7-in. lip, as shown by Fig. 34. The effective lip consisted of the top flange angles, the bottom angles of the lip which were extended about 4 ft. over the main portion of the girder, and two 6-in. ship-building channels, also extended 4 ft. over the main portion of the girder. The end shear is 36 000 lb. for

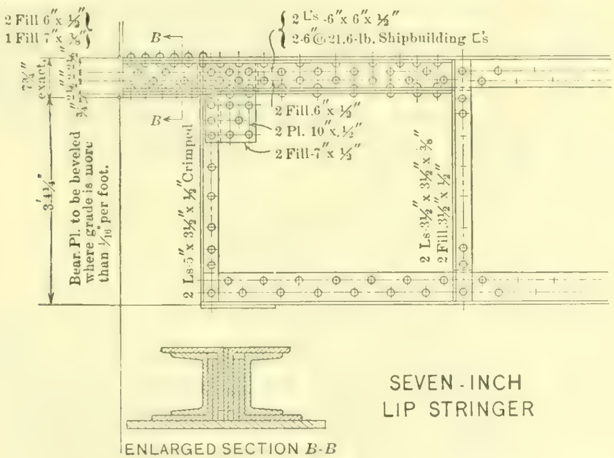


FIG. 34.

a 45-ft. span, which is the standard length, and the section modulus of the combined lip section is 58.6. Assuming the load on the lip to be applied 12 in. from the main portion of the girder, the maximum stress in the lip is 7 500 lb. This, however, is only true if the rivets connecting the different sections of which the lip is constructed are sufficient to make them act together as a unit. The rivets in the web portion of the lip were not sufficient for this purpose, as they were too close to the neutral axis. The channel section, therefore, was used, in order to be able to utilize the rivets in the horizontal flanges to take up the shear, and as the flanges in a standard 6-in. channel were too narrow to use ⅞-in. rivets, the ship-building channel was selected.

As the head-room under the structure at the 14th Street Station did not permit the construction of a mezzanine station, the "hump" type of express station was used. At this station, prior to the reconstruction, there existed center-track girders, but they were removed, as they interfered with the construction of the new bents, and were not used in the new structure, as the 7-in. lip construction of these girders was not desirable in the new work. Fig. 35 shows the details of the new structure carrying the express track. Lips of increasing depth were used to raise the track stringers to the new grade at the beginning of the approach. When the stringers reached above the cross-girders, pedestals, riveted to the ends of the stringers, were used, being intended at the same time to stiffen the stringers longitudinally. When the pedestals reached a height of about 5 ft., two-post bents were used for supporting the track stringers. These bents were stiffened horizontally in pairs, as shown, and were held from lateral overturning by plates riveted to the bottom of the posts and extending through the space between the twin cross-girders to connection with the vertical web member of the cross-girders.

The bents in front of the express station platform were provided with plate brackets for the support of the inside express platform girders; the outside platform girders were carried on columns, as shown by Fig. 36, extending through the lower-deck platforms, and supported on the lower-deck cross-girders. The deck construction of the platform was made very shallow, as at the other hump stations, in order to keep the level of the upper deck as low as possible.

The 86th Street Station was also a hump station. In the original scheme, no express station was to be built at this location, but, after all the steel for the new track stringers, which were designed with 7-in. lips, to be set directly on top of the existing cross-girder, had been fabricated and delivered, the Public Service Commission ordered an express station to be built there. The general details of the steel structure were similar to those at 14th Street, except that seats were provided on top of the supporting bents to carry the lip ends of the track stringers as fabricated.

The 42d Street Station had originally two side-platforms. As the head-room from the street to the under side of the structure could be made sufficient to place a mezzanine station by raising the existing track construction about 1 ft., this type of station was decided on

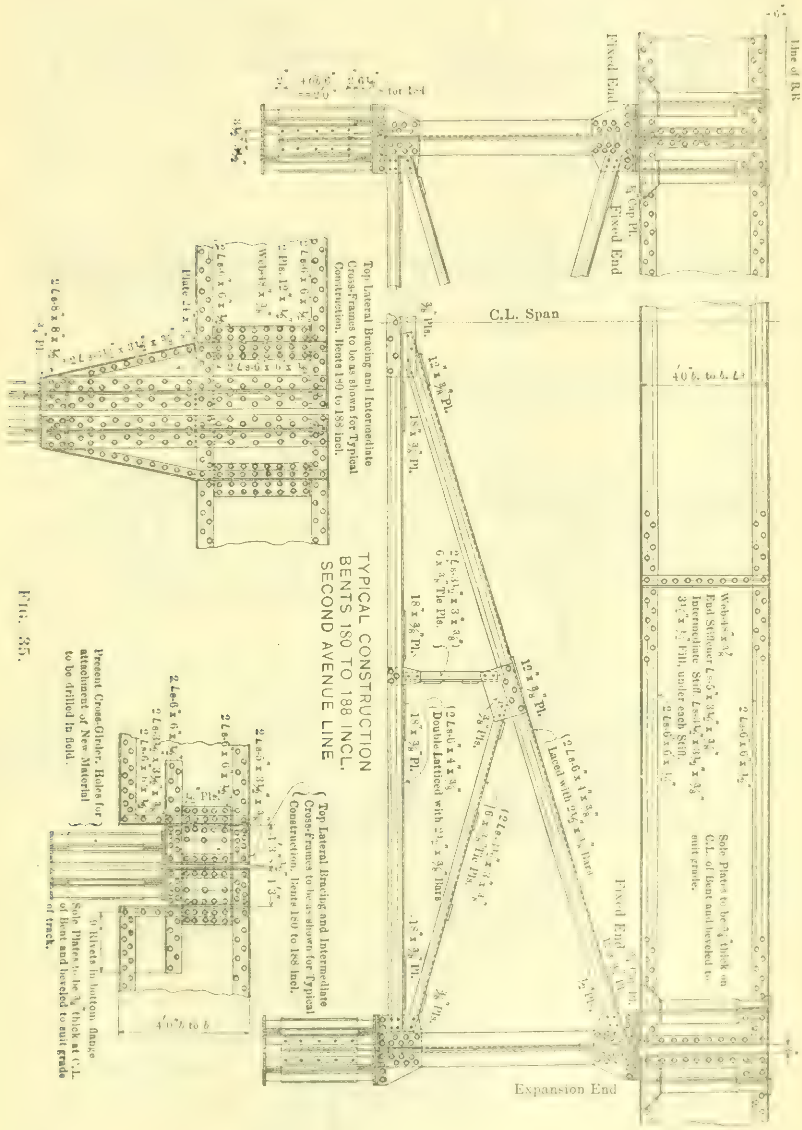

TYPICAL CONSTRUCTION
BENTS 180 TO 188 INCL.
SECOND AVENUE LINE
C.L. Span
Fixed End
Fixed End
Expansion End
Top Lateral Bracing and Intermediate Cross-Frames to be as shown for Typical Construction. Bents 180 to 188 incl.
Top Lateral Bracing and Intermediate Cross-Frames to be as shown for Typical Construction. Bents 180 to 188 incl.
Present Cross-Girder. Holes for attachment of New Material to be drilled in field.
Sole Plates to be ¾" thick on C.L. of Bent and beveled to suit grade.
Line of R.R.

FIG. 35.

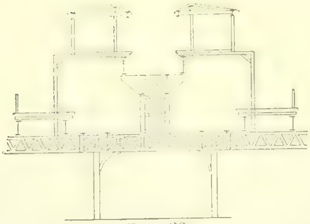


FIG. 36

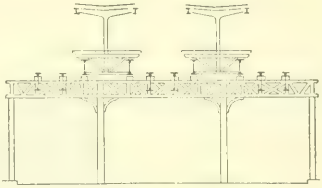


FIG. 37

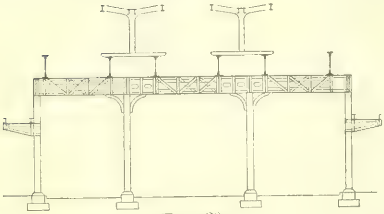


FIG. 38

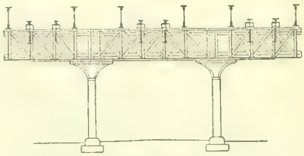


FIG. 39

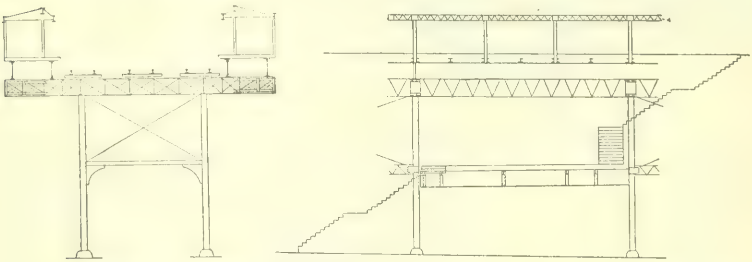


FIG. 40

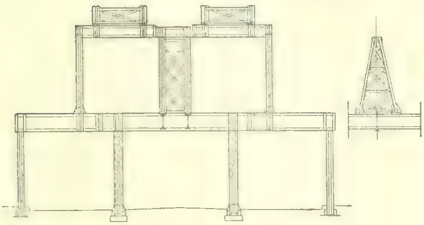


FIG. 41

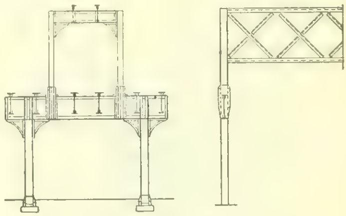


FIG. 42

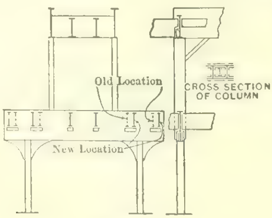


FIG. 43

at this point. The raising of the track structure was accomplished without disturbing the existing columns and cross-girders, by supporting the lips of the track stringers on seats, resting on top of the cross-girders. As the new arrangement of the station required considerably greater width of structure than formerly, the cross-girders were extended at both sides, the extensions being carried by columns from the sidewalk near the curb line; in other words, after the reconstruction, all the cross-bents, carrying the station structure, were four-column bents, except one, which was in the center of 42d Street, where the traffic conditions prohibited the placing of additional columns. Before the reconstruction, a few of the bents, which supported the old station buildings, had the cross-girders extended to the sidewalk, where they were supported by columns; the old sidewalk columns were generally retained, but the cross-girder extensions were renewed in order to carry the new track loads, as shown by Fig. 37. The new extensions were made a twin-girder construction in order to provide supports for the old track stringers which were shifted to the new location. In order to prevent secondary stresses in the cross-girder, the top chords of the existing cross-girders were cut above the street columns, so as to make the cross-girder act as three spans of simply supported girders.

Fig. 38 shows one of the bents in the span, where the mezzanine station was placed. As the existing cross-girder was not sufficiently strong to carry the additional loading, which included the mezzanine station, a new twin cross-girder, divided into three pieces at the points of supports, was constructed. The portions of this girder over the central supports were made with solid webs, in order to get sufficient rivets to carry the shear, and, to make the section open for painting, etc., holes were cut in the webs, as shown on Fig. 38. The existing street columns were retained for the support of this bent, but the sidewalk columns were renewed in order to make connections for brackets to support the new stairways leading to the mezzanine. The other bent, supporting the mezzanine station, was in the center of 42d Street and, therefore, could not be provided with more than two supporting columns. Fig. 39 shows the details of this bent. New and heavy columns with over-all dimensions of 15 by 18 in. were used to support the new twin cross-girders, which were made 6 ft. deep in order that they would be strong enough to carry the loading on the

cantilevers, 16 ft. 6 in. long. The track was carried on through bridge girders, which also supported the platforms and the suspended mezzanine station.

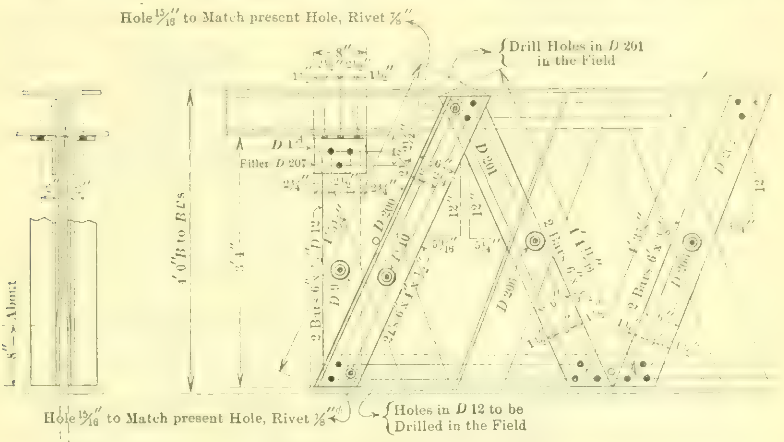
At 92d Street the structure is high above the street. The old station had a center platform, and the station building was at platform level. Access to the station was obtained by stairs at both sides of the street. The stairs were connected by a bridge under the track structures, and a stairway led from this bridge to the station building. As the station building and the center platform were in the way of the new center track, the station was reconstructed with side platforms and a mezzanine station building, before the existing station structure was removed. Fig. 36 shows the construction. The twin cross-girders were lengthened to support the new platform girders, and the latter were set on top of the cross-girders, and carried the new platform construction. For the support of the mezzanine station, two girders were framed into the Phœnix columns supporting the structure, one on each side of the street in the span, when the mezzanine station was built. As it was not considered advisable to disturb the existing riveting of the Phœnix columns, beveled fillers, with holes cut at the locations of the rivet heads, were added between the flange of the column and the splice-plates, connecting the girders to the column.

The purpose of double latticing the existing center-track stringers was to reduce the bending stress in the top flanges by shortening the distance between the panel points, and to reinforce the connection at the end lip, which, experience has shown, is the weakest point of the stringer. The general method of reinforcing is shown by Fig. 44 and is the standard reinforcement for track stringers of this type used prior to this work. The method of end reinforcing is not entirely satisfactory, as the rivets are in tension, but, nevertheless, it acts as an additional safeguard.

Details of Design.—Section No. 5-A.—Section No. 5-A included the Third Avenue Line between 116th and 129th Streets and in 129th Street between Third and Second Avenues. The work involved the reconstruction of the station at 125th Street for express service and the connection of the express track with the upper deck of the Harlem River Bridge. North of 125th Street Station it was desired to retain the existing center track on the lower level, as it was used as the main north-bound track, while the outside track was used as an

approach track to the Company's Yard at 129th Street. It became necessary, therefore, to carry the over-head track high enough to provide sufficient head-room for the trains below, and as it was desired to keep the over-head track approximately level through the station, this height was maintained to the south end of the over-head platforms.

The approach to the over-head track commenced at a point near 121st Street. As there had been a center-track structure prior to the reconstruction, the general type of construction was similar to that described under Section No. 3, for the northerly approach to the 42d Street Station, with tapered girders, seats, and towers supporting the



REINFORCEMENT OF STRINGERS
SECOND AVENUE LINE

FIG. 44.

over-head tracks. Fig. 41 shows a cross-section of the upper-deck structure at the station. On account of the upper-deck track being carried higher above the grade of the lower deck than at 42d Street, it was possible to carry cross-girders above the local tracks to support the express platform girders. These cross-girders are supported at the inside by the towers carrying the track structure and at the outside by columns resting on the lower-deck cross-girders.

North of the 125th Street Station, where the lower-deck center track was retained, the distance between the tracks was increased by moving the supporting track stringers sidewise with the tracks to make room for the columns of two-column bents, set on the top of the lower-deck cross-girders to carry the upper-deck track structure.

Fig. 42 shows the details of these bents. The column section was made only 12 in. wide, in order to make the sidewise shifting of the tracks as small as possible. The columns were spliced to the supporting lower-deck cross-girders, and extended to the top of the upper-deck cross-girders, which were riveted to the column shafts. The upper-deck track stringers were of the lip type, resting on top of the cross-girders, and the bottoms of the stringers were flush with the bottoms of the cross-girders. Longitudinally, the bents were provided with stiffening trusses extending from the top of the columns to within about 6 ft. of the lower track level, so as to leave a clear walking space below the trusses.

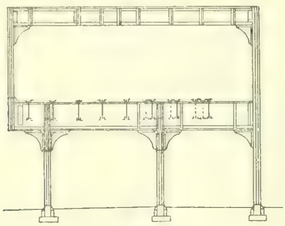


Fig. 45

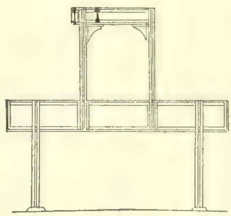


Fig. 46

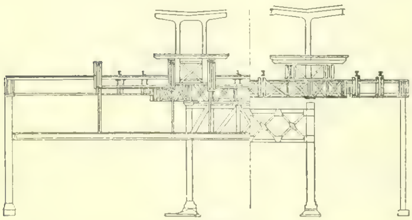


Fig. 47

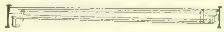


Fig. 48

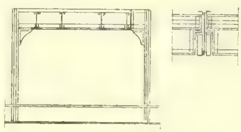


Fig. 49

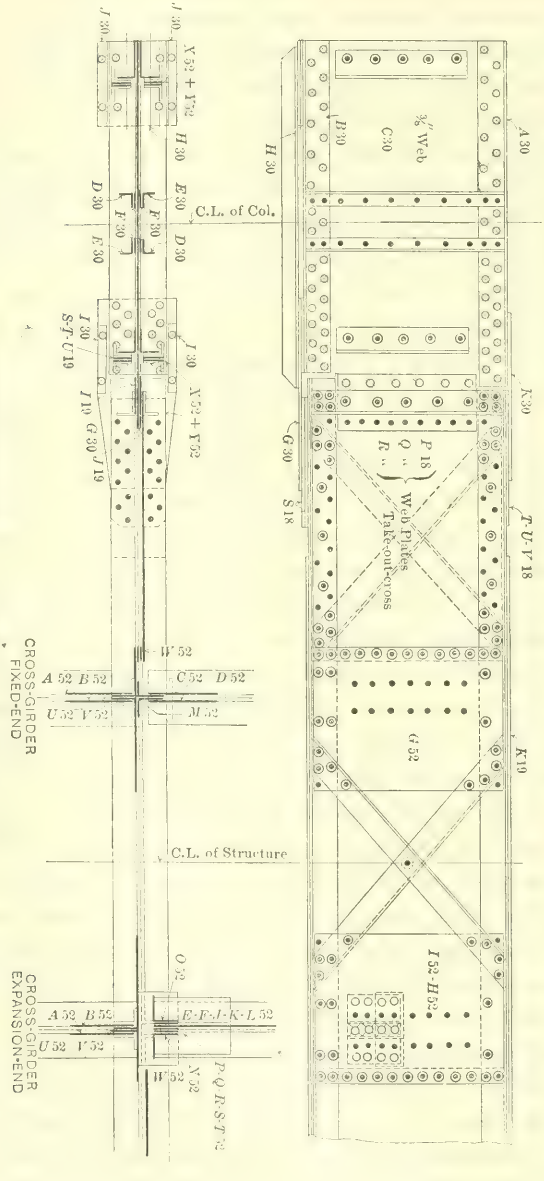
At 128th Street, the presence of a cross-over on the lower deck prevented the placing of the columns for an upper-deck bent on the lower cross-girder, and the upper-deck structure, therefore, was carried a distance of about 90 ft. between the supports. A through bridge construction was used for this span, as shown by Fig. 43, in order to get sufficient depth of the longitudinal girders. Fig. 43 shows also a cross-section of one of the supporting bents. As the column section could not be materially widened, on account of the necessity of maintaining the proper clearance between the side of the car and the column, the column section necessary for the increased vertical loading and wind stresses was obtained by increasing the column shaft lengthwise.

The through bridge girders were supported directly on top of the columns, and the secondary track stringers were framed into the floor-beam girders. The track stringers of the adjacent spans, which were of the standard lip type, were supported on seats attached to the web of the end floor-beam girders.

The other bent, supporting the through bridge girders, is shown by Fig. 45. On account of the lower-level track layout, the columns supporting the upper-deck structure had to be placed outside of the lower-deck tracks. They were made 15 in. square in cross-section, and one of them was carried down to the street, making the lower-deck bent a three-column bent. This was made necessary because the tracks of the lower deck at this point curved into the 129th Street Yard, and, therefore, were considerably outside of the regular street columns. The other column was supported by the cantilever end of the new lower-deck cross-girder.

From the south end of the 125th Street Station to a point north of 128th Street the existing columns and cross-girders were replaced with a new structure on account of the added loading, as shown by Figs. 42 and 45. As reinforcement for these girders had been designed and fabricated ready to place prior to this reconstruction work, it was first intended to use this material, with such additional material as might be required, for reinforcing the existing cross-girders to carry the additional load. It was found, however, that the weight of the reinforcing material greatly exceeded that of a new girder, and that the field work would be considerably greater; it was decided, therefore, as previously stated, to use new girders. South of the 125th Street Station the cross-girders were reinforced, as shown by Fig. 50. The track stringers of the lower deck were remodeled at the ends to connect to the reinforced and new cross-girders. Riveted connections were made at the fixed points, and seats were provided at the expansion points, as shown by Fig. 50.

North of the 125th Street Station, where the outside track girders were shifted, the procedure was first to erect the new cross-girders with the stringers in the original position, and then to shift the stringers to their new position. The cross-girders were provided, therefore, with temporary seats for the girders in the old position, as well as with permanent seats.



CROSS-GIRDERS-992 TO 1001 INCL. THIRD AVENUE LINE

Fig. 50.

The existing structure in 129th Street was sufficiently strong to carry the additional loading of the upper level track without reinforcement. At the westerly end of 129th Street, the existing structure carried a wide platform of the 129th Street Station. The columns supporting the upper-track structure were carried through this platform to the lower-deck cross-girders, to which they were spliced. East of the platform the column locations were determined by the track arrangement of the lower deck. The general details of the bents were similar to those of the bents in Third Avenue, north of the 125th Street Station, except that the stringers were supported on seats attached to the web of the cross-girders and that the latter, in several cases, cantilevered at the one end over the column and supported the one line of track stringers on the cantilever end, as shown by Fig. 46. At the east end of the structure, the new track divided into two to connect with the upper-deck tracks of the Harlem River Bridge.

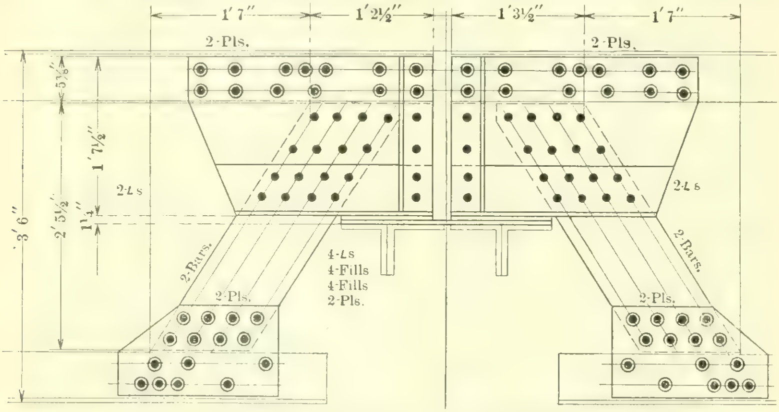
Details of Design.—Section No. 5-B.—Section No. 5-B covered the structure of the Second Avenue Line in Second Avenue between 116th and 129th Streets. The existing structure carried three tracks between 116th Street and the entrance to the Yard at 129th Street. Beyond this point there was a two-track connection to the existing Harlem River Bridge Stations at 121st and 127th Streets. The existing type of structure was as shown by Fig. 30 and described under Section No. 4-A, consisting of Phœnix columns, twin-lattice cross-girders, and lattice track stringers supported on the cross-girders by lips, 7 in. deep.

The new plan involved replacing the existing station at 127th Street with a new express station at 125th Street, the connection of the express and local tracks with the upper deck of the new Harlem River Bridge, and the connection of the reconstructed local tracks with the lower deck of the Harlem River Bridge and with the Yard.

The existing structure at 125th Street was low, but, in order to make proper connection to the upper deck of the Harlem River Bridge, it was raised sufficiently to make it possible to construct the new station at 125th Street as a mezzanine station. All three tracks were raised, starting at a point near 122d Street, and, at 125th Street, the rise amounted to about $7\frac{1}{2}$ ft.

Fig. 51 shows the construction used to raise the track at the beginning of the grade. The ends of the track stringers were remodeled, and the sizes of the supporting lips were gradually increased until a

point was reached where the track stringers rested entirely on top of the cross-girders. After this point was reached the existing cross-girders were raised with the track stringers, which necessitated replacing the existing columns with new and longer ones, but did not involve any change in the track stringers. Fig. 52 shows the details of the new columns and their longitudinal and transverse bracing, which extends from the top of the columns to within about 14 ft. of the street level. The column section was the standard section used for the improvement works. On account of the wide base of the support of the cross-girder on the column tops, and the stiffeners of the cross-girder, no splicing of the columns to the cross-girder was necessary.



REMODELING OF CENTER TRACK LONGITUDINAL GIRDERS
AT BENT 892

FIG. 51.

As the columns had to be erected under cross-girders in place and over existing foundations with anchor-bolts in place, a detachable type of base was used, but, on account of the sub-surface conditions, they differed somewhat from those already described. There was a 12-in. gas pipe and also a 12-in. water main at the west side of Second Avenue, almost directly under the columns. The old foundations, which were retained, were divided at the top into three parts, with the gas and water mains passing between the three portions of the foundations. The old columns had cast-iron bases, into which the column shafts fitted and were connected by rust caulking. Some of the cast-iron bases were shaped to bridge across the openings in the foundations, and others rested on top of grillage beams spanning the openings.

The detachable parts of the new bases were made of a similar shape to fit the anchor-bolts, but they were of cast steel, and were connected to the fixed portion of the base by six bolts, as shown by Fig. 52. The bases on the east side of the street, where the foundations were built in one piece, were made shorter, but, for the sake of uniformity, of a similar construction.

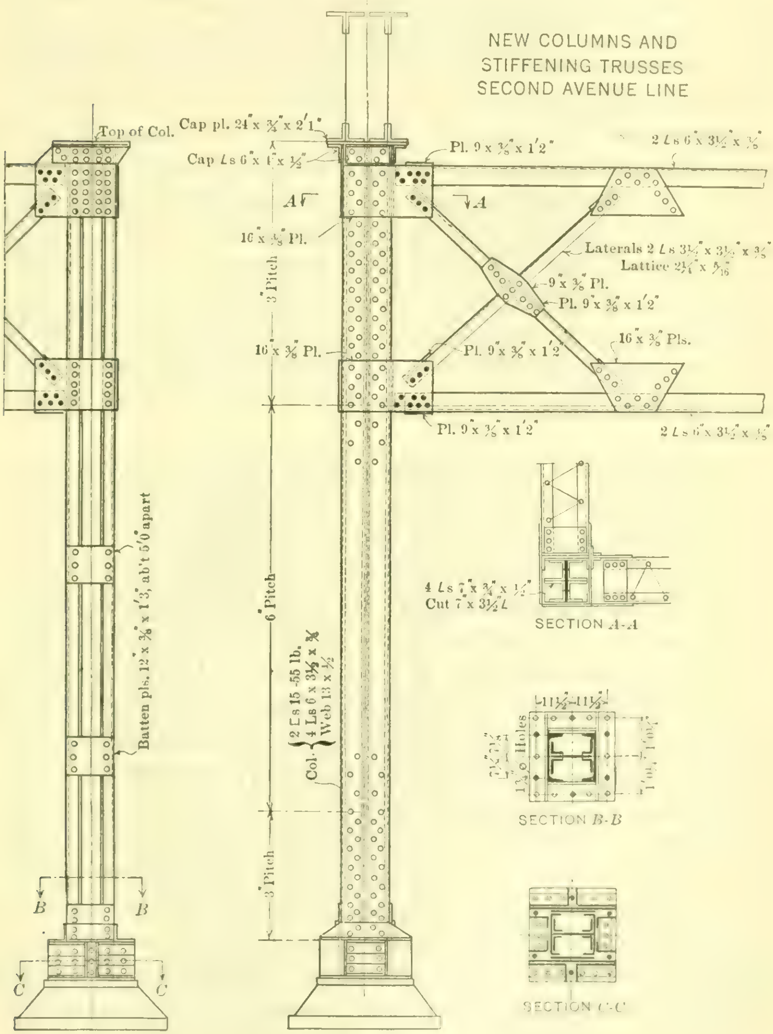


FIG. 52.

The width of the structure had to be increased throughout the 125th Street Station on account of the additional space needed for the platforms. This was accomplished by adding new pieces to both ends of the cross-girders, supported at one end by seats attached to the existing cross-girders and at the other end by columns on the sidewalk, near the curb line. At 125th Street, where the curvature of the curb around the street corners prevented the placing of the sidewalk columns on line with the other sidewalk columns, the cross-girder extensions were lengthened, and the columns were placed along the curb line in 125th Street on line with the Second Avenue building line.

Fig. 53 shows the details of the seats and the cross-girder extensions. The existing twin cross-girders had flange angles turned back to back, and each had four angle end stiffeners. Plates were inserted between each set of stiffener angles extending beyond the end of the girder, and the seat was attached to these plates; the cross-girder extensions rested with a lip on these seats where the existing track stringers were to be supported on these extensions, the tops of the extensions were made flush with the existing cross-girders, and wide seats were made at the proper places for the lips of the track stringers to rest on.

The mezzanine station was placed in the span across 125th Street, and this span was 63 ft. long. As usual at mezzanine stations, the whole construction was carried by through bridge girders in this span, as shown by Fig. 54. For the purpose of obtaining better end details, the cross-girder extensions in this span were set with the top flanges higher than the main portion of the cross-girders, and the end lips of the outside through bridge girders, which rested on the extension, were, therefore, somewhat different from those of the other through bridge girders. The center through bridge girders, which also acted as platform girders, had end lips nearly the full width of the flanges of the supporting twin girders on end seats, so as to distribute the load uniformly over the twin girders. The outside through bridge girders, which carried a load very much smaller than the other girders, were supported by a shallower flange on the cross-girder extensions. The adjacent track stringers of the outside tracks were remodeled and carried on seats on the girder extensions. The through bridge floor consisted of built-up floor-beams and secondary track stringers consisting of pairs of I-beams framed into the floor-beams.

Outside of the mezzanine span, the platform stringers consisted of plate girders supported on top of the track stringers. Fig. 47 shows a cross-section of the structure at the through bridge span and outside

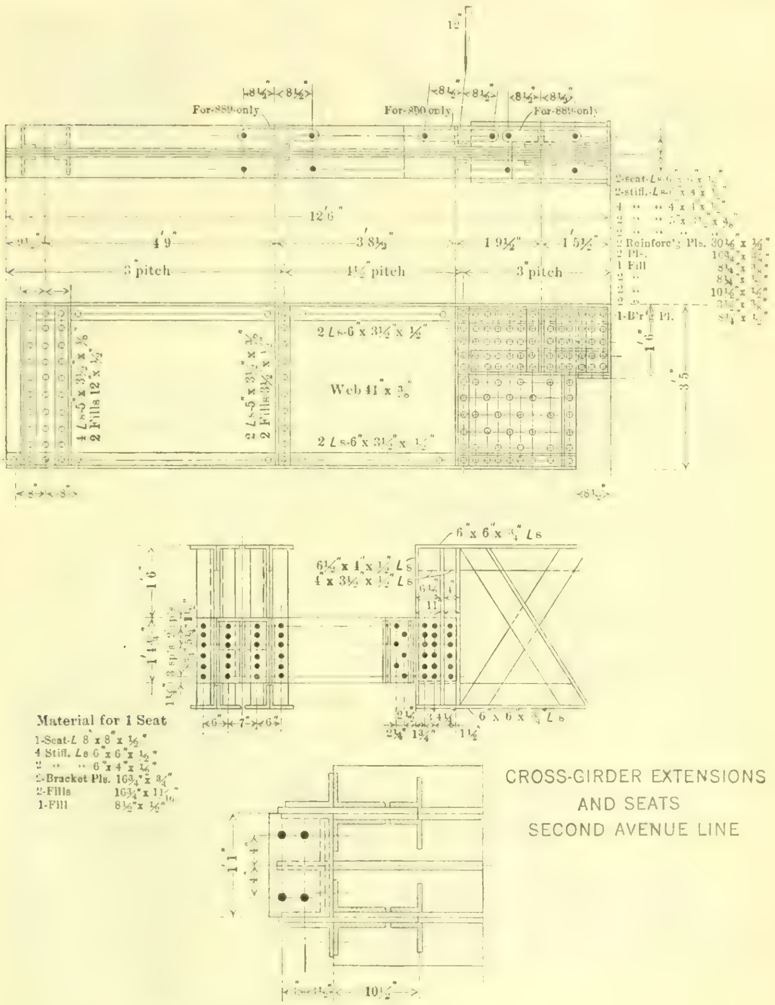
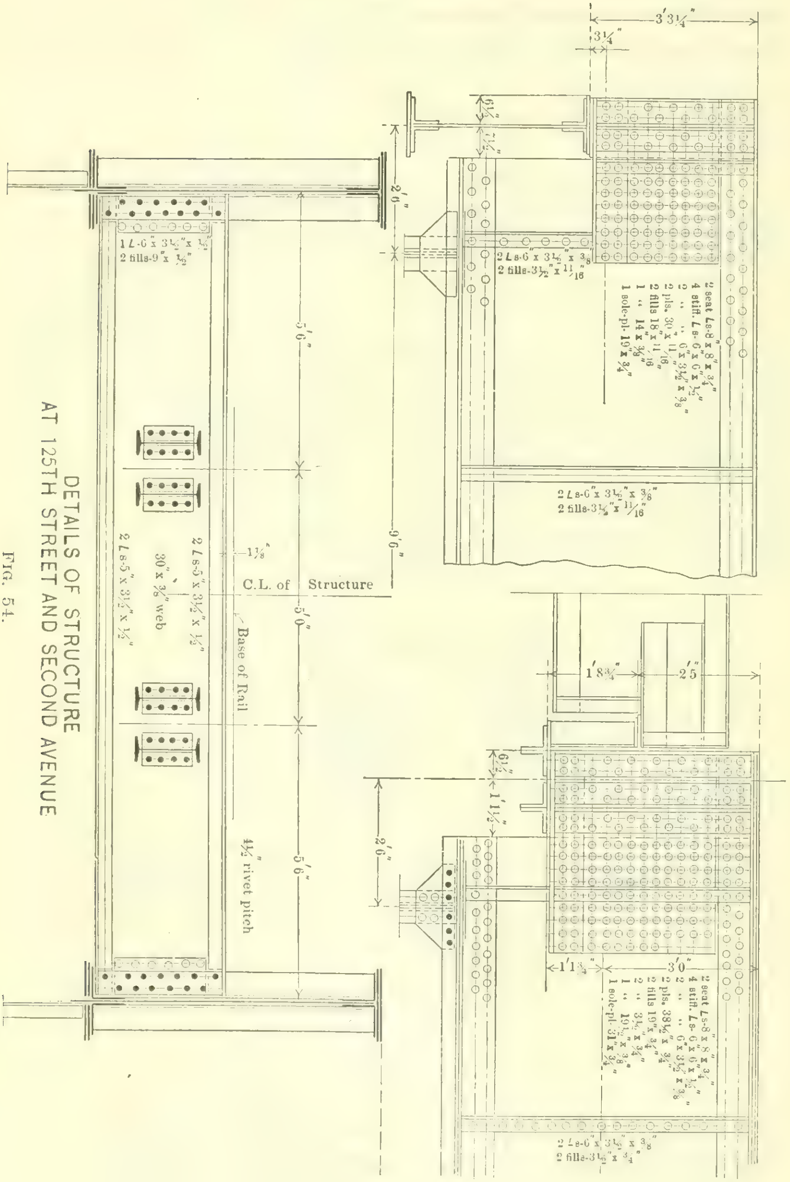


FIG. 53.

of this span. On account of the heavy load on the columns in the through bridge span, they were built up of the following material: two 18-in., 60-lb. channels, four 6 by 4 by ¾-in. angles, and one 13 by ¾-in. web-plate.



DETAILS OF STRUCTURE
AT 125TH STREET AND SECOND AVENUE

FIG. 54.

North of the 125th Street Station the three-track line was changed into four tracks, with the two center tracks ascending to the level of the upper deck of the Harlem River Bridge and the two outside tracks descending to the original level just south of 128th Street where connection was made with the 129th Street Yard. The details of the lower-deck reconstruction were similar to those south of the 125th Street Station. The upper-deck structure, as far as the yard crossing, consisted of two-post bents set on top of the lower-deck cross-girders, braced laterally and longitudinally. At one point, where the outside lower track turns under the new upper-deck structure, the distance between the track levels required very shallow track girders, as shown by Fig. 48. In order to gain as much depth as possible, these girders were tapered, the top flange following the grade of the upper-deck track, and the lower flange being horizontal.

North of the yard crossing, the lower-deck tracks were carried under the upper-deck tracks in order to obtain as easy curves as possible at the connection to the Harlem River Bridge. Fig. 49 shows a typical bent supporting the structure at this location. As the span was wide the cross-girders were made deeper, extending above the top of the track stringers, which rested on seats attached to the cross-girder webs. The bents were braced longitudinally for the full length of the columns and laterally by plate brackets. At the bottom, the columns were spliced to the supporting cross-girders.

Details of Design.—Section No. 5-C.—Section No. 5-C comprised the bridge, carrying the Second and Third Avenue Lines across the Harlem River at 129th Street. The old bridge (Fig. 56) had three spans, supported on five piers carried down to an elevation of 32 ft. below mean high water, and resting on a solid bed of sand and gravel. The old bridge, which carried two tracks and two footwalks, was a pin-connected structure, built in 1887; since that time it has been in continuous operation, and was in good condition when removed. The center span was a swing span, supported on a drum bearing, and turned by a rope drive by hydraulic power.

Fig. 55 shows a cross-section of the drum. The drum consisted of two 12-in. channels connected to a sleeve over the center pin by shallow built-up girders. It supported the bridge structure at six points, and was itself supported on rollers running on a circular track and held in position by tie-rods and angles. To the outside

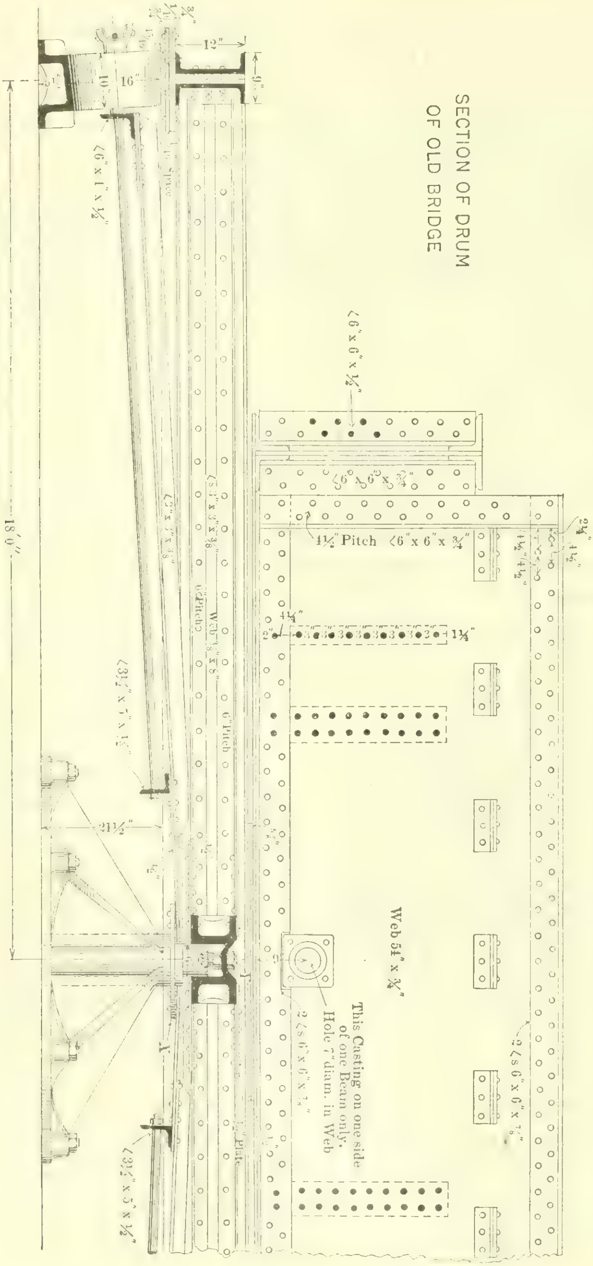

SECTION OF DRUM
OF OLD BRIDGE
Web 54" x ¾"
This Casting on one side
of one Beam only.
Hole 7" diam. in Web
1½" Pitch ∠6" x 6" x ¾"
18' 0"
21½"
12"
16"

FIG. 55.

ends of the axles of the rollers was attached a ring with a semicircular groove in which was laid the cable that turned the bridge. This cable went around about three-quarters of the ring, and was carried up to the under-side of the operating tower over vertical sheaves, set tangential to the ring. The cable was then carried over sheaves that changed the direction to the horizontal, so that the cable could be connected to the horizontal hydraulic ram under the floor of the operating tower. It will be noted that the speed of the cable determines the horizontal speed of the rollers, but that the speed of the drum is determined by the circumferential speed of the rollers, and the drum, therefore, travels at a speed equal to twice that of the cable. It may be stated that the arrangement worked satisfactorily, and that the bridge could be opened very quickly, but that, on account of the shallow drum, the bearing on the rollers was not uniform and caused them to wear out.

The new track facilities, on account of the Manhattan Elevated Improvements, involved the construction of a double-deck four-track structure in place of the existing two-track bridge, but it was necessary to maintain traffic, except during short periods, over the two existing tracks. It was decided, therefore, to build an entirely new bridge at a location where the work would not interfere with the operation of the existing bridge, and, when the erection of the new bridge was finished, to replace the old spans with the new spans complete. The method of doing this work is described later under the heading, "Erection."

Fig. 57 shows the new bridge. The length of the end spans, from center to center of piers, was 103.28 ft., and the length of the swing span was 248.4 ft. over all. The clear openings of the swing span were 103.7 ft. The new bridge was built as a pin-connected structure. Fig. 86 shows the construction of the north span.

The new swing span was center-supported. It was at first intended to design the new bridge as rim-supported, similar to the old bridge, but it was found that the depth of the new drum, needed in order to avoid undue deflection, would necessitate the removal of a considerable portion of the center pier, and this could not be done without interfering with the operation of the bridge for a considerable length of time. The casting for the center support of the new bridge, as designed, necessitated the removal of a portion of the pier, but this

work could be done without interfering with the traffic, as hereafter described.

Plate XLV shows the center casting and the arrangement of the operating machinery. The weight of the bridge was carried to the casting through three disks, on which the bridge turned. The center disk was made of phosphor bronze and the other two were steel forgings.

The turning machinery was driven by two 40-h.p. electric motors, and two other 40-h.p. electric motors were provided for the operation of wedges and rail-lifts. After the machinery had been received and erected, and while the swing span was still resting on the pile foundation on which it was erected, an attempt was made to work the operating machinery. It was then found that the bearings for the beveled gears were not sufficiently stiff under the stress, and that the gears were thrown out of line and became bound. The cause of the trouble was that the bearings for the beveled gears were cast and supported independently of one another. A new set of castings was designed, therefore, with two bearings for the corresponding gears in the same casting, and after they had been placed no further trouble was experienced.

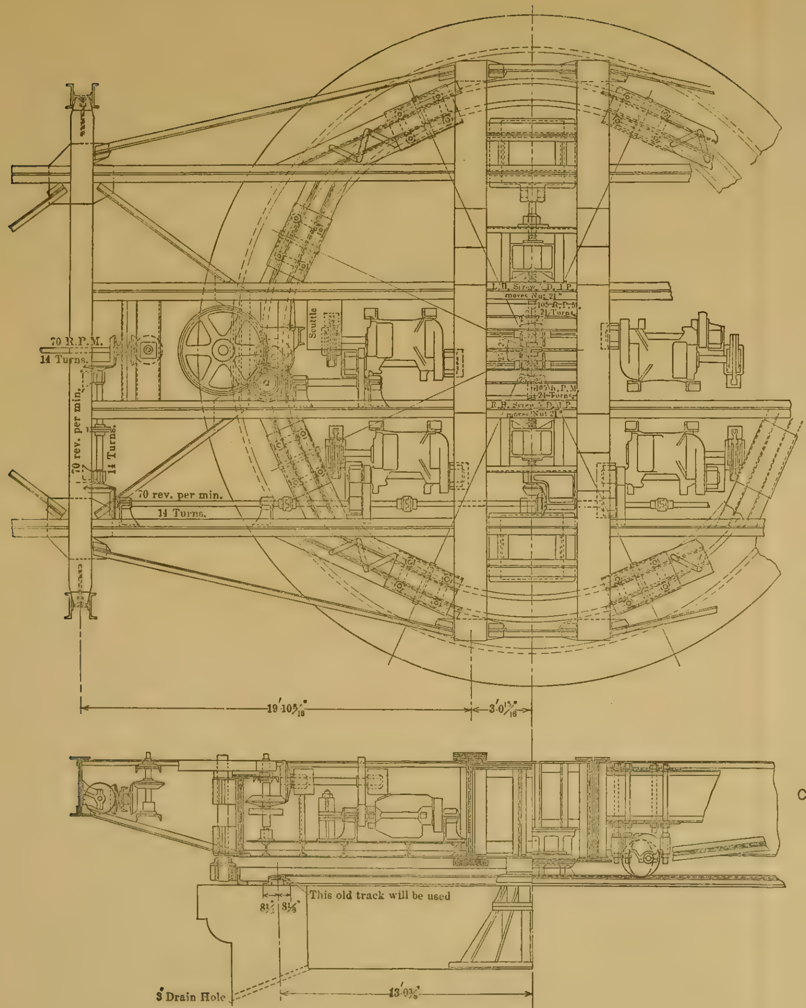
The method of determining the live-load stress in the swing span deserves to be stated. When closed, the bridge is supported on three bearings, and is, therefore, statically indeterminate. The usual way of determining the stress, by the three-moment method, which assumes uniform moment of inertia throughout the truss, was evidently not satisfactory, as the moment of inertia varies greatly, and it was found by comparing the stresses obtained in this manner with those determined by the accurate method hereafter described, that the stresses by the first method were in error as much as 100 per cent.

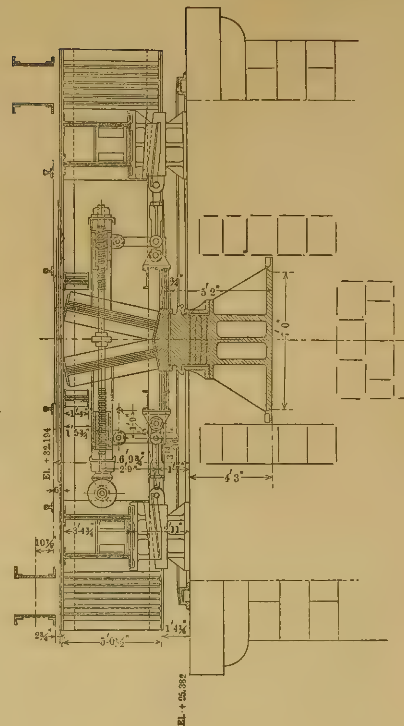
The method used was developed for this purpose by F. W. Gardiner, M. Am. Soc. C. E., and is described as follows:

Consider the member, *H-m* (Fig. 58), cut at the center; the truss will then be two single spans, and will be statically determinate. Apply to the cut ends of *H-m* two equal and opposite forces just sufficient to hold the ends in contact. The stresses in the truss members for any condition of loading will then be the combined stresses computed as a single span and the stresses induced by a pair of equal and opposite

FIG. 56.—OLD HARLEM RIVER BRIDGE.

FIG. 57.—HARLEM RIVER BRIDGE. NEW SWING SPAN.





PLAN AND SECTIONS SHOWING
CARRYING GIRDERS, PIVOT, AND OPERATING MACHINERY
AT CENTER PIER OF SWING SPAN.

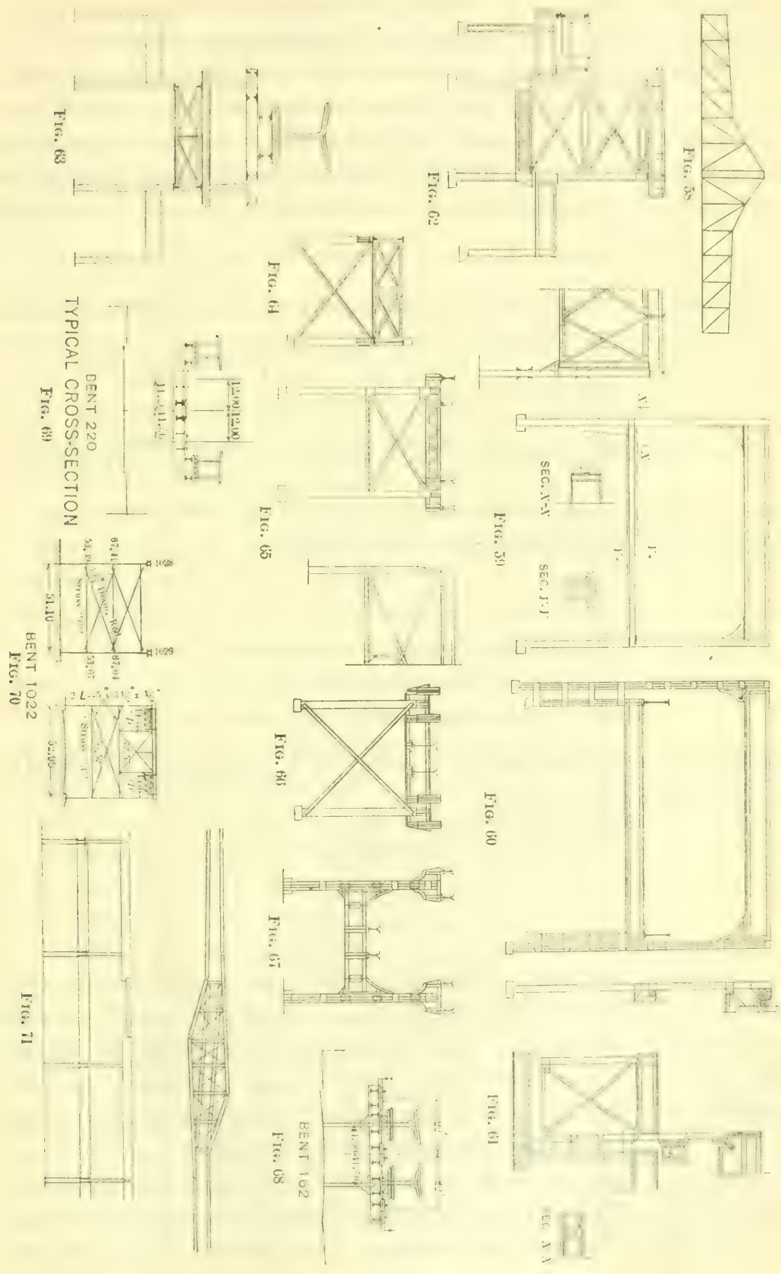


Fig. 58
Fig. 59
SEC. X-X
SEC. Y-Y
Fig. 60
Fig. 61
SEC. X-X
Fig. 62
Fig. 63
Fig. 64
Fig. 65
Fig. 66
Fig. 67
BENT 162
Fig. 68
BENT 220
TYPICAL CROSS-SECTION
Fig. 69
BENT 1022
Fig. 70
Fig. 71

forces applied at the cut ends of *H-m,* and just sufficient to hold the cut ends in contact for the position of loading to be computed.

In order to determine the forces to be applied to the cut ends of *H-m,* it is first necessary to determine the distance, D; a pair of equal and opposite forces of unity will bring the ends of *H-m* together; let this distance be D. Then, if, for any condition of loading, the horizontal displacement of the cut ends of *H-m* is Δ, the force which will hold the cut ends of *H-m* in contact will be $\frac{\Delta}{D}$.

To find D, consider half the bridge with a horizontal force of unity applied at the cut end of *H-m.* The horizontal movement of this end will be $\frac{D}{2}$, and the work done will be $\frac{D \times 1}{4}$.

The internal work will be $\sum \frac{s' d}{2}$, where s' is the stress induced in the members by unity horizontal force at the cut end of *H-m*, and d is the elongation or compression of a member;

$$d = \frac{s' L}{a E},$$

where $L =$ length of a member,
$a =$ area of a section,
$E =$ modulus of elasticity.

Therefore, the internal work is equal to $\frac{s'^2 L}{2a E}$, and, making this equal to the external work, we have,

$$\frac{D}{4} = \frac{s'^2 L}{2a E},$$

or,

$$D = 2 \frac{s'^2 L}{a E}.$$

It will be sufficient to find the reactions for a unit load at each panel point, and we could put a unit load at each panel point, compute the horizontal separation of the cut ends of *H-m,* divide the separation by D, giving the horizontal force necessary to hold the ends together, and find the reactions resulting from the combined action of the unit panel load and the horizontal force at *H-m.*

The work can be much simplified at this point by the use of Maxwell's "Reciprocal Theorem." This theorem, applied to the bridge in question, states that the horizontal displacement of the cut end

of *H-m*, due to a vertical load of unity at any panel point, is equal to the vertical displacement of the panel, due to a horizontal force of unity at the cut end of *H-m*.

This theorem can be proved as follows: Let S = the stress in the members due to a unit load at a panel point, and let S' = the stress in a member due to a unit horizontal force at the cut end of *H-m*. Then, the vertical displacement of the panel point, due to a unit horizontal force at the cut end of *H-m*, equals $\frac{SS' \ L}{a \ E}$, and the horizontal displacement of the cut end of *H-m*, due to a unit vertical load at the panel point, equals $\frac{S' \ SL}{a \ E}$, therefore, the vertical displacement of the panel point due to a unit horizontal force at *H-m* is equal to the horizontal displacement of *H-m* due to a unit vertical load at the panel point.

The extension or compression of all the members for a unit horizontal force at the cut end of *H-m* has been found in computing the distance, *D*, and a displacement polygon is drawn. We know that Point 6 is fixed, and that the displacement of Point 0 must be horizontal; we, therefore, give 0 a vertical displacement by revolving the diagram about Point 6, until Point 0 comes in horizontal line with 0. In doing this, we have displaced the other panel points in a direction perpendicular to their radius with Point 6 as a center, and in amount proportional to their distance from Point 6. The vertical displacement of the panel points are taken by scale from the diagram, and these, in accordance with Maxwell's "Reciprocal Theorem", are equal to the horizontal separation of the cut ends of *H-m* due to a unit load on each panel point, respectively.

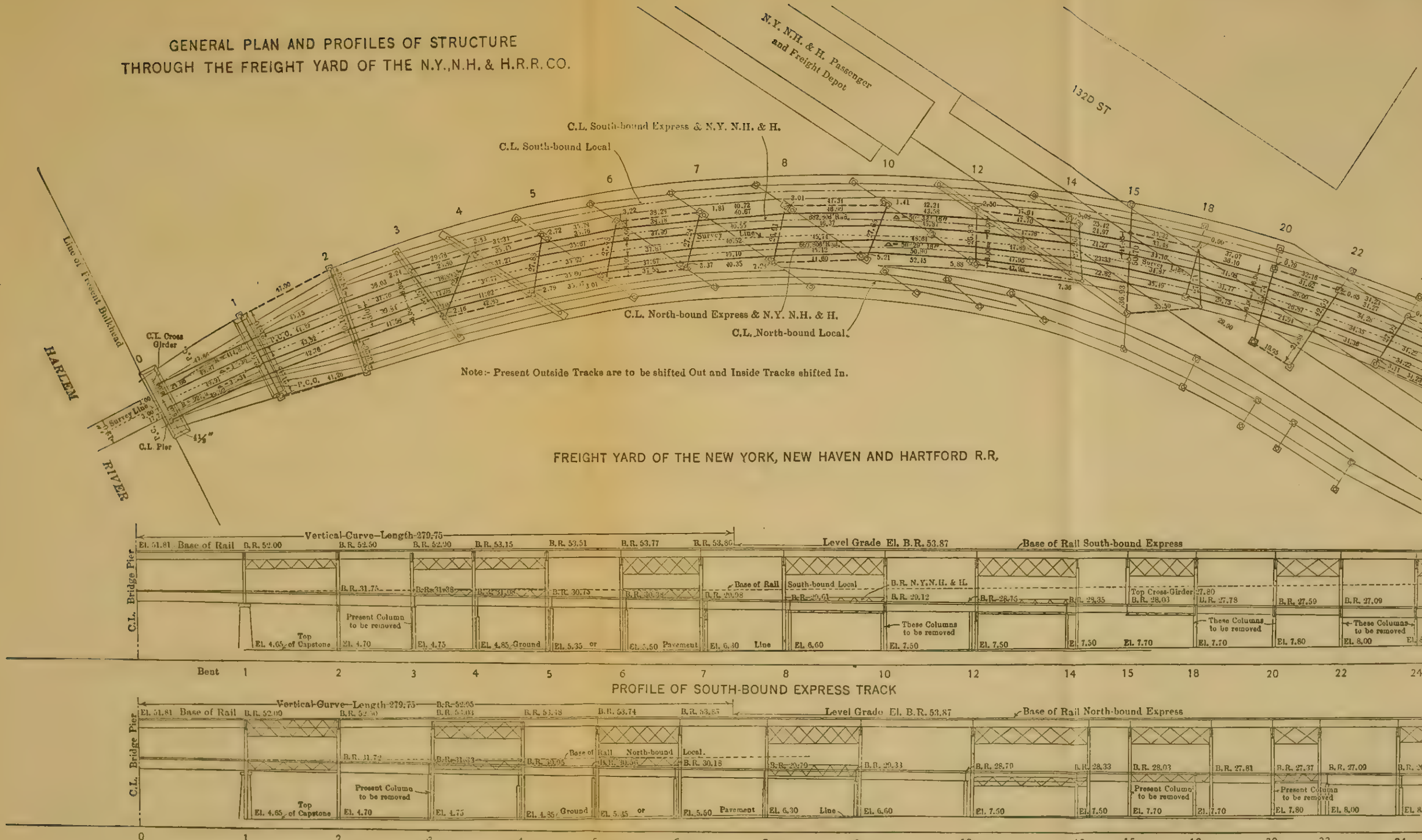
Details of Design.—Section No. 5-D.—The existing structure between the Harlem River and 133d Street, which comprised Section No. 5-D, was built partly over the freight yard of the New York, New Haven and Hartford Railroad and partly over the Interborough Rapid Transit Company's Yard. The portion over the New York, New Haven and Hartford Railroad Company's yard was a four-track structure; the two outside tracks were the main-line tracks and the two inside tracks, which gradually descended and at the north side of the yard were carried under the main-line structure, were used for shuttle service between the station at 129th Street and Third Avenue and

the New York, New Haven and Hartford Railroad Passenger Station at Willis Avenue. North of the Railroad Company's yard the existing structure carried only the two main tracks, and through the yard the existing columns were placed so as not to interfere with the tracks and platforms.

The new structure consisted of two additional tracks connecting with the upper deck of the Harlem River Bridge. Through the New York, New Haven and Hartford Railroad Company's Yard they were directly above the shuttle tracks, and north of the yard they were continued between and above the outside main tracks. Fig. 72 shows the general plan and profiles through this section. It may be noted here, that the curved track layout and the skew location of the bents throughout this section necessitated a specially careful survey of the existing structure, and involved very extensive calculations to determine the lengths and locations of the members forming part of the new structure. It was found that the work was much facilitated and more easily checked by using analytical geometry in the calculations. A system of rectangular co-ordinates was used; the zero point and the location of the axes were chosen arbitrarily, but in such a manner that all the co-ordinates were positive, but as small as possible, so as not to have unnecessarily large figures to compute. The equations of all the cross-girders of the old structure were then calculated, as well as those of the new tracks, cross-girders, etc.; also, the co-ordinates of the points of intersection, determining the lengths and locations of the new structure members. This method of calculation was very effective, and had the important advantage that errors were not carried from point to point, and when discovered were easily corrected.

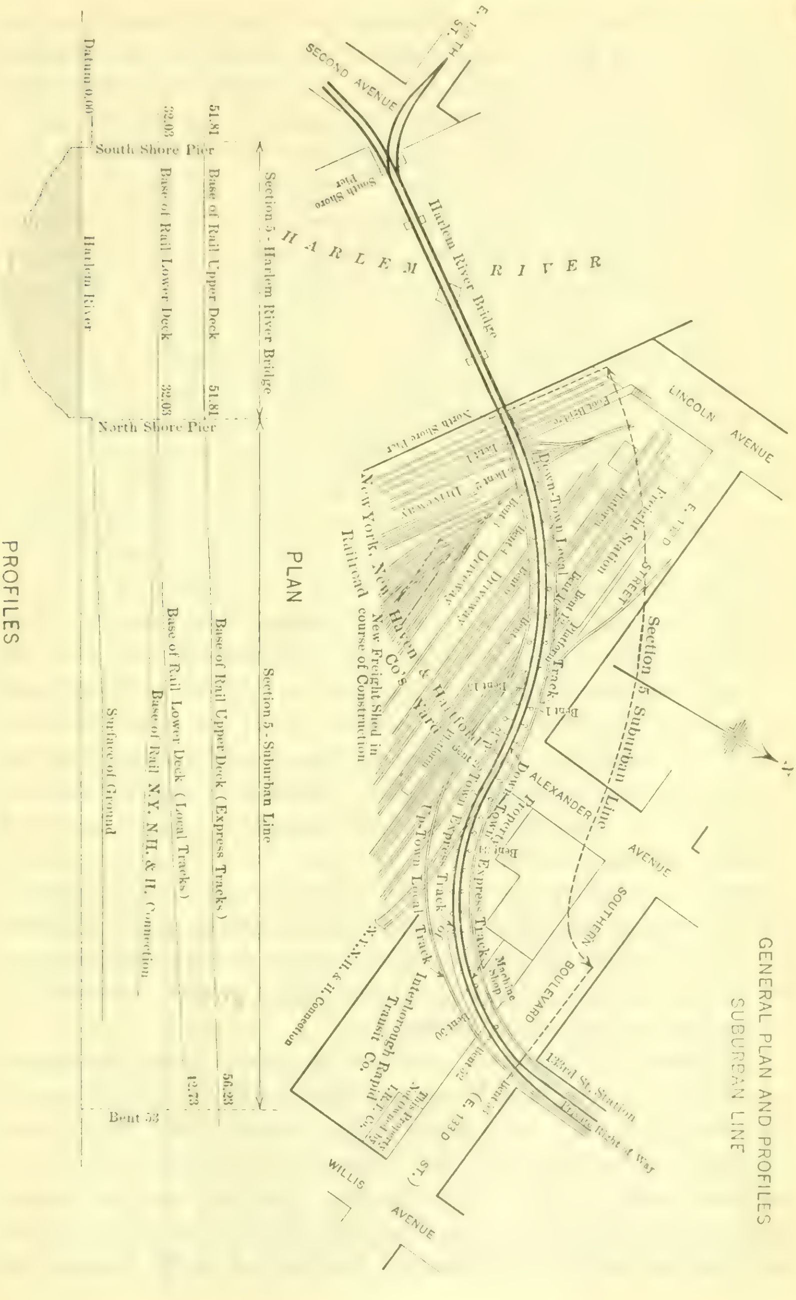
Plate XLVI shows the alignment of the old and new structures through the New York, New Haven and Hartford Railroad Company's Yard. The first bent of the old structure was supported on a masonry pier. The columns, supporting the new structure, were placed one at each end of the old pier, as the track spacing of the lower deck did not allow sufficient room between the tracks for the supporting columns. In order to brace laterally the column shafts, which were more than 48 ft. long, a yoke, consisting of 15-in. channels, was fastened to the columns at the level of the cap-stone of the pier, and secured to the cap-stone by expansion bolts, as shown by Fig. 59. The columns

GENERAL PLAN AND PROFILES OF STRUCTURE THROUGH THE FREIGHT YARD OF THE N.Y., N.H. & H.R.R. CO.



PROFILE OF SOUTH-BOUND EXPRESS TRACK

PROFILE OF NORTH-BOUND EXPRESS TRACK



GENERAL PLAN AND PROFILES
SUBURBAN LINE

Fig. 72.

of the second bent of the new structure also had to be placed outside of the existing tracks, on account of the necessary car clearance on the lower deck, as shown by Fig. 60. The new columns were braced laterally to the existing columns, and longitudinally the columns of this bent and those of the first bent were braced together by two stiffening trusses between each pair of columns, one truss below the track level of the lower deck and the other at the top of the column, so as not to encroach on the side clearance around the curved tracks.

Fig. 61 shows the general type of construction of the following bents. The existing structure was not sufficiently strong to carry the additional loading, and, therefore, the columns supporting the new structure were brought down to the surface. The new columns had to be centered between the existing lower-deck tracks, in order to get the necessary side clearance, and had also to be kept on line with and within the construction width of the lower-deck bents, so as not to encroach on the side clearance of the yard tracks. The existing columns were only 12 in. wide, and it was necessary, therefore, to make the new ones of the same width, and, in order to get a sufficiently strong column section, the new columns were made of a box section, as shown. The new bents were not made on line with the old ones, but crossing from one old bent to the next so as to make the new cross-girders as short as possible. Where the old cross-girders were in the way of the new columns, the cross-girders were cut to make room for the columns and were supported by seats on the new column shafts. The old columns were generally retained and tied together with the new ones.

Owing to the fact that it was decided to support the new columns on the concrete foundations of the existing columns, as will hereafter be explained, the load of the new structure was transferred to the existing column foundations by foundation girders, connected to the column by diaphragms, as shown.

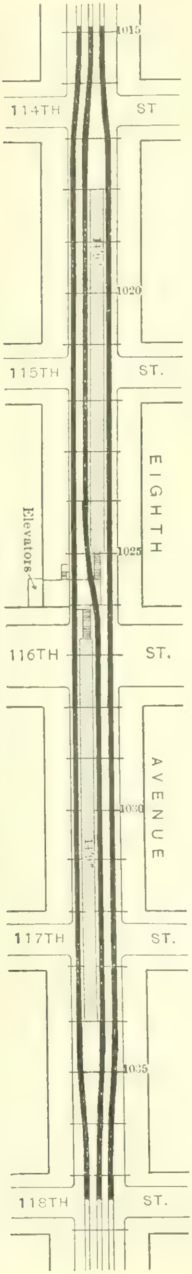
Just south of 132d Street, which is only partly opened for public traffic, the New York, New Haven and Hartford Railroad has a freight platform with a track on each side. The old bent (Bent No. 15) was supported on three columns, one centered on the platform and the two other outside of the adjacent tracks. The new bent was placed on line with the old one. The lower-deck bent was entirely renewed, partly on account of the additional loading and partly because the

New York, New Haven and Hartford Railroad Company desired to have the head-room under the structure increased, in order to make room for overhead wires for electric traction, which were being put up in the yard. The west column, supporting the overhead structure, was centered between the lower-deck tracks, and was supported on the lower-deck cross-girder, because it could not be carried down to the surface, where it would have interfered with the surface track.

The other bents in 132d Street were similar to this one in general details. The lower-deck track stringers were remodeled, where they crossed the surface tracks (Fig. 73), in order to get sufficient head-room for the electric traction system. The remodeling was accomplished by riveting new flange angles to the girder webs at the desired height and cutting away the portion of the girders below the new flanges. The stringer ends were changed to rest on seats provided for them on the new cross-girders.

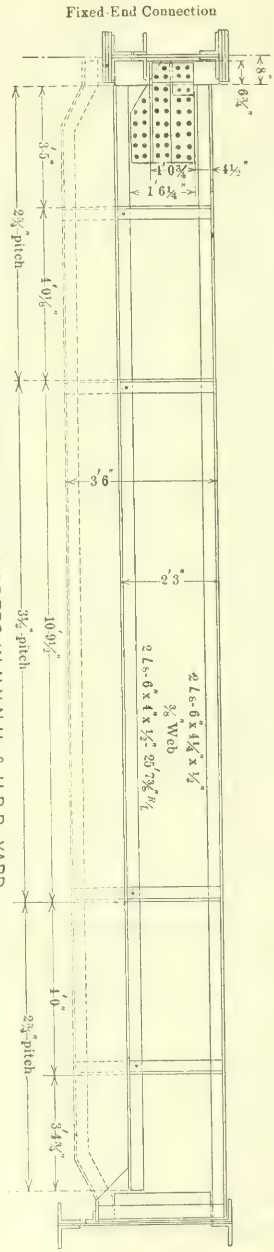
Between 132d and 133d Streets, the existing structure carried only two tracks, as the shuttle tracks at the Willis Avenue Station turned east through 132d Street. The existing tracks between 132d and 133d Streets were a considerable distance apart, and were supported separately on tower constructions. The two new tracks were supported together on similar tower constructions. The columns of the bents on the outside of the curved tracks were set at an angle with the vertical, in order to provide for the horizontal thrust due to the centrifugal force.

The existing structure across 133d Street was supported by two four-column bents, one near the curb line on each side of the street, with a span length of about 64 ft. between them. The new upper-deck structure was supported on bents placed on a line with the existing bents, and with the columns extending to the street, taking the place of the two inside columns of each bent, as shown by Fig. 62. The old cross-girders were supported by seats on the new columns, and the columns were cross-braced transversely. The new track structure was supported between the bents by deep lattice girders carried by seats attached to the columns and acting as longitudinal stiffening trusses as well as supporting girders. The deck was carried on top of these girders, and consisted of shallow cross-girders and secondary track stringers, made up of two I-beams, framed into the cross-girders.



116TH STREET STATION, NINTH AVE. LINE

FIG. 74.



REMODELING OF GIRDERS IN N.Y.N.H. & H.R.R. YARD

FIG. 73.

Details of Design.—Section No. 6-A.—Section No. 6-A extended from 133d to 147th Street. The portion from 133d to 145th Street is on a private right of way, 50 ft. wide; the remainder is in Third Avenue. The existing structure was a two-track line, and, through the right of way, consisted of track stringers supported on masonry piers, except at street crossings, where they were supported on column bents. There were stations with center platforms at 133d, 138th, and 143d Streets. The new work included the addition of two overhead tracks from 133d Street to a point north of 143d Street, where the two tracks joined and ran as a single track down to the grade of the lower-deck track at 145th Street, and continued on that grade as an additional center track. The new track layout also included a single-track ramp running from the upper deck at 138th Street to the lower deck at 141st Street, connecting at each end with both main tracks. At 143d Street provision was made for turn-outs of both the new upper-deck tracks to connect with a new extension between this line and the existing elevated portion of the subway at Brook and Westchester Avenues. New overhead island platforms were added to the stations at 133d, 138th, and 143d Streets, and the existing island platforms on the lower deck were entirely rebuilt. Prior to the erection of the new structure, the existing south-bound track was moved away from the north-bound track to make room for the new structure, as will be described later.

Fig. 63 shows a cross-section of the structure at the 133d Street Station. The new upper-deck structure, as well as the widened lower-deck platform, was carried on new column bents, about 40 ft. from center to center. The lower-deck platforms were supported on two deep stringers carried by seats on the columns, as well as by a center stringer framed into and supported by cross-frames between the columns and between the centers of the deep stringers, so that the span of this center stringer was only about 20 ft. The upper-deck structure had both platforms and tracks supported on columns spliced to the lower-deck columns and also by intermediate columns supported on top of the deep platform stringer, making the spans of the upper-deck track structure only about 20 ft., which was desired, in order to make the construction height and the distance between the two platform levels as small as possible. Fig. 64 shows the details of the lower-deck bent. The columns were of the standard cross-section, 15 in. square, and braced transversely. The channels of the column

shafts were turned with their webs parallel to the bent, and the columns, therefore, were stiff enough to resist longitudinal stresses without additional bracing. The column bases were riveted to the columns, and designed to resist the bending stresses; they were fastened to the foundations by four anchor-bolts.

The construction at the 138th Street Station is similar to that at the 133d Street Station. Between these stations, the new structure consisted of two sets of track stringers supported on column bents which, in every third span, were braced longitudinally to form a tower to resist the longitudinal forces; the bents between the towers were designed for vertical and transverse loads only.

Fig. 65 shows the new structure at the crossing of 135th Street. The supporting columns were centered under the tracks, and were braced both horizontally and transversely from the top to a point about 14 ft. above the street level. The cross-girder was supported on the top of the column, and its center portion, which did not carry any load, but acted as transverse bracing, was designed with a lattice web; the end portions, which carried the load, had solid webs.

North of 138th Street, the center ramp, leading from the upper to the lower deck, commenced. Fig. 66 shows the supporting cross-bent near the top of the ramp. The details of the bent are similar to those shown by Fig. 65, except that the cross-girder is a plate girder throughout. The outside track stringers are carried on the top of the cross-girder, and the center-track stringers by seats attached to the web of the cross-girder. Fig. 67 shows the cross-bent at an intermediate point of the ramp. On account of the necessary clearance for cars on the center track, the column tops could not be braced transversely. The column section, therefore, was strengthened by increasing it in width and making the flanges of four angles and a cover-plate. The web-plate stopped about 4 ft. 6 in. from the top of the column, and, in its place, was inserted a twin bracket the web of which supported the track stringers. When the center track was sufficiently low, the column tops were braced transversely by an arched plate girder. Between the 133d and 138th Street Stations part of the column bents were also braced longitudinally to form towers.

The same construction was used between 144th and 145th Streets, where the new structure carried only a single track. From 145th Street to the end of the section at 147th Street, the new work consisted

simply of adding center-track stringers to the existing structure, and replacing the old cast-iron column bases with new cast-steel bases.

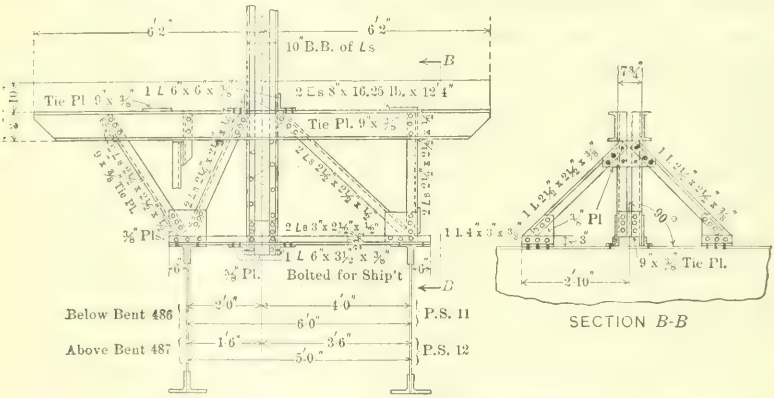
Details of Design.—Section No. 6-C.—Through the greater portion of this section, which extended, on Third Avenue, from 147th Street to Fordham Road, the existing columns were replaced. These columns were only 12 in. square, and consisted of two 12-in. channels latticed together. The new columns were built up of two 15-in. channels, four angles, and a web-plate, but were only made 12 in. wide, as the franchise rights of the line did not permit wider columns. The existing columns were bolted to shallow cast-iron bases, which were fastened to the granite caps of the foundations, the latter being 18 in. thick. The only weak member for the added loading was the column. The foundations were retained, and the new column bases, where replaced, were generally similar to the existing ones. The new cast bases, however, were enlarged, and made of steel instead of iron. Bolt holes were provided to match the existing anchor-bolts, and, in addition, four other bolt holes were provided, to be used in case the existing anchor-bolts were broken or damaged. At the top of the column a fixed connection to the cross-girder was made in the usual manner by splicing the column to the stiffeners of the cross-girder.

The existing station at 149th Street had a center platform and a side platform on its east side. The new station, which was reconstructed for express service, has two center platforms and a mezzanine at the north end of the platforms. Fig. 68 shows a cross-section of the station as remodeled. The old cross-girders, which were plate girders, were retained, but the flanges were reinforced with cover-plates, and cantilever extensions were added at both ends, on account of the additional width of the structure. The new extensions were spliced to the old girders, and seats were provided for all new track stringers. New stiffeners were added over the columns to support the additional loading. On account of the method of erection used, the old track stringers were retained in place and used for the support of the new platforms. The new platform structure is shown by Fig. 75. The platform beams were made of two channels each, and were supported on frames of light angles and stiffened against overturning by angle braces to the supporting stringers.

The stations at Tremont Avenue and Fordham Road were also reconstructed for express service with two island platforms. The

general details of the steelwork were similar to those of the 149th Street Station.

The existing stations at 156th, 161st, 166th, 169th Streets, Claremont Parkway, 174th, and 183d Streets, had center platforms. All these stations were reconstructed with side platforms, so as to make room for the express track. Fig. 69 shows a typical cross-section of the remodeled structure, and Fig. 76 shows the details of the extensions added to the existing cross-girders in order to provide supports for the side platforms. The station building was generally placed on a mezzanine and was connected with the platforms by stairs at the outside edge of the platforms. As it was desired to support the added structure without the use of additional columns, the extensions to the cross-girders were made long enough for this purpose, producing cantilevers of the cross-girders about 22 ft. long.

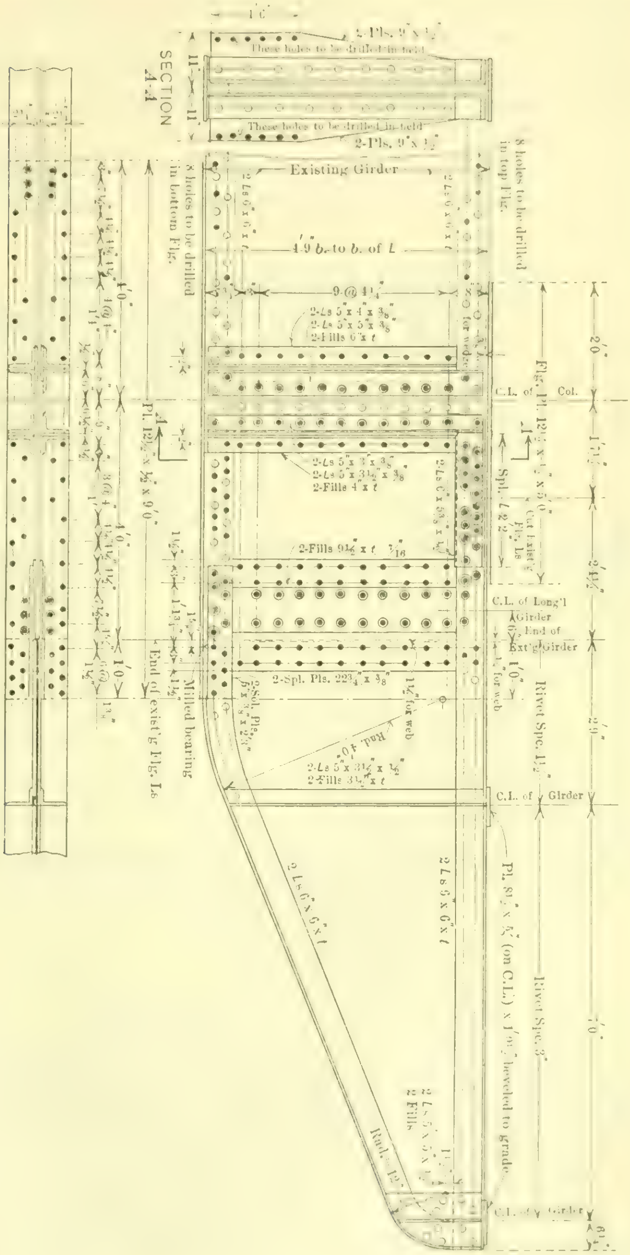


CANOPY COLUMN AND PLATFORM, THIRD AVENUE LINE

FIG. 75.

Details of Design.—Section No. 7.—The work under Section No. 7 included the reconstruction of the stations of the Ninth Avenue Line at 66th, 116th, 125th, and 145th Streets for express service.

The existing structure at 66th Street was similar to that in Third Avenue, and carried three tracks before the reconstruction. At both these stations the structure was low, and, therefore, the express stations were made of the "hump" type. On account of the added loading, the lower-deck cross-girders and columns were replaced with new material throughout the length of the new platforms at 66th Street; but at 145th Street, where the cross-girders were twin girders and the

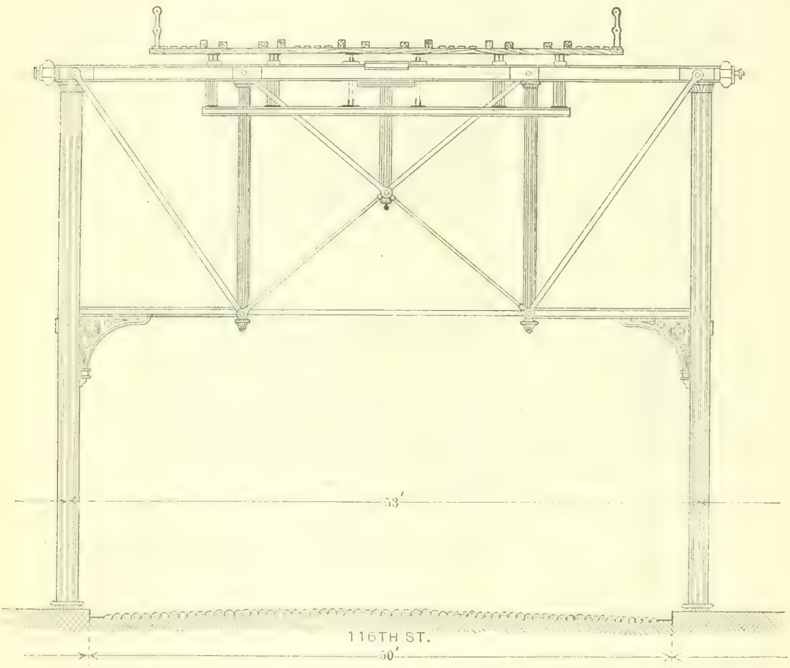


CROSS-GIRDERS, THIRD AVENUE LINE

FIG. 76.

columns Phœnix sections, the existing structure was sufficiently strong to carry the added loading. The general details of these two stations are the same as those described for the "hump" stations on the Second and Third Avenue Lines, and will not be repeated here.

At the 116th Street Station the existing tracks were nearly 50 ft. above the street surface. The track stringers were lattice girders with 7-in. lips, resting on top of cross-trusses, supported on columns placed



ORIGINAL STRUCTURE
ON EIGHTH AVE.
AT 116TH STREET.

FIG. 77.

on the sidewalks, as shown by Fig. 77. The columns were six-section Phœnix columns, and the trusses were built of flat tension members and Phœnix column compression members, except the top chord, which was a box section, built of channels and plates. The station at 116th Street had two island platforms lying end to end between the two existing tracks, separated only by the space needed for a station building at the end of each platform and a flight of stairs leading

from each station building to a common passageway immediately below the track structure and leading to an elevator building at the corner of Eighth Avenue and 116th Street. South of the south platform and north of the north platform, the structure carried three tracks.

The reconstruction work consisted of moving the south platform to the west and the north platform to the east, so as to make room to carry the center track through and past the platforms, as shown by Fig. 74, which change, of course, necessitated also the shifting of the local tracks.

The added loading made it advisable to stiffen the column shafts, so as to reduce their bending stresses. The cross-truss, which rested with the top chord on the top of the columns, was about 20 ft. deep, and its bottom chord tied the columns together about 30 ft. above the street level. Longitudinally, the columns were braced in pairs by two struts, one near the top and one at the level of the bottom of the cross-truss, and two diagonal tie-rods. To this was added some additional bracing, both transversely and longitudinally, as shown by Fig. 70, extending down to within about 14 ft. of the street level. The longitudinal struts were made fish-bellied, to conform to the existing ones, and the transverse struts were made straight. Both were fastened to the columns by a single sleeve with collars around each column. The sleeve was fastened to the column by four bent plates riveted to the column flanges between the existing flange rivets, for the heads of which slots were made in the bent plates. Web-plates and flange angles were riveted to the outstanding legs of the bent plates, and the horizontal legs of the flange angles were riveted together with plates, to which again were riveted small shaped plates fitting around the column shafts, so as to form a collar to prevent tension in the column rivets. The stiffening trusses were riveted to these sleeves and collars. Longitudinally, the strut was supplemented with a cross of round bars, which, at the bottom, went through a special steel casting, riveted to the strut, and, at the top, was fastened through a yoke, riveted to the old strut. The cross-strut was supplemented with a diagonal bracing of angles which, at the top, were connected to the column flanges by bent plates, as shown.

By shifting the track stringers sidewise, so as to make room for the center track, the stresses in the top chord would be increased, and,

therefore, it was decided to reinforce it. As it was not desired to do the reinforcing in such a manner that shoring should be needed during the work, the method shown by Fig. 70 was adopted. The portion to be reinforced was that between the column and the first vertical post. A lattice box-girder of the same width as the top chord was suspended from it by two yokes, one at each end. Plates were then driven in between the reinforcing girder and the top chord, and secured with rivets.

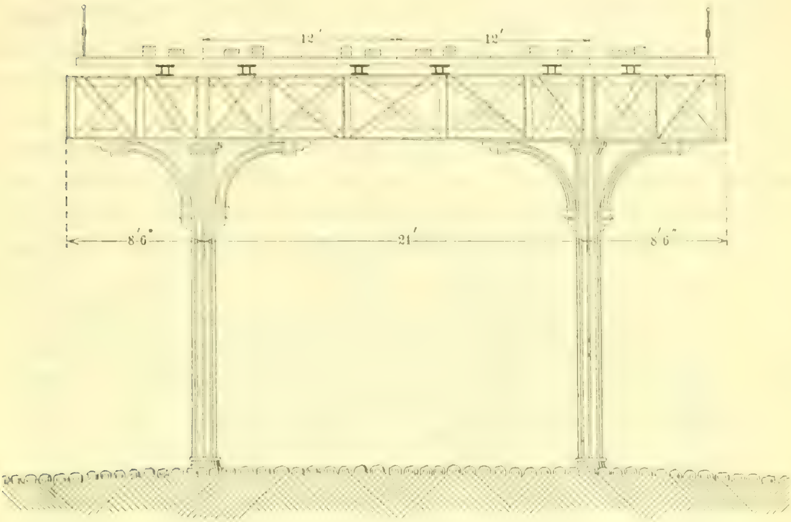
In addition to the reinforcing of the columns as described, it was originally contemplated to reinforce the connection of the columns to the foundations, but, as this was found to be unnecessary, it was not carried out.

A new mezzanine station was constructed under the structure at the level of the existing footbridge leading to the elevator building. This new station was carried by two new twin girders, connected to the columns of adjacent bents in a manner similar to that previously described for connecting the transverse and longitudinal struts.

The existing structure at the 125th Street Station was similar to that on the Second Avenue Line, and is shown in cross-section by Fig. 78. The same arrangement of platforms existed here as at the 116th Street Station, and the new arrangement was similar to that described for that station, except that the station buildings were retained. As the existing cross-girders were not long enough to carry the full width of the new structure, short extensions were added to them at each end, with seats to carry the outside stringers supporting the local tracks in their new position.

Fig. 79 shows the details of the cross-girder extensions and the new track stringers. Two vertical plates were riveted to the flange angles of the lower girder and extended beyond it the full width of the new seats. Two vertical diaphragms stiffened these plates, one at the cross-girder end and the other at the center of the seats. Angles were riveted to the top and the bottom of the vertical plates, and were connected to the top and bottom flanges of the cross-girder by horizontal splice-plates. Regular seats were provided on the vertical plates for the new track stringers. The track stringers were lattice girders, in order to conform to the existing stringers.

South of the 125th Street Station, where St. Nicholas Avenue crosses the elevated structure, were two spans, one of about 99 ft.



ORIGINAL STRUCTURE
ON EIGHTH AVE.
AT 125TH STREET.

FIG. 78.

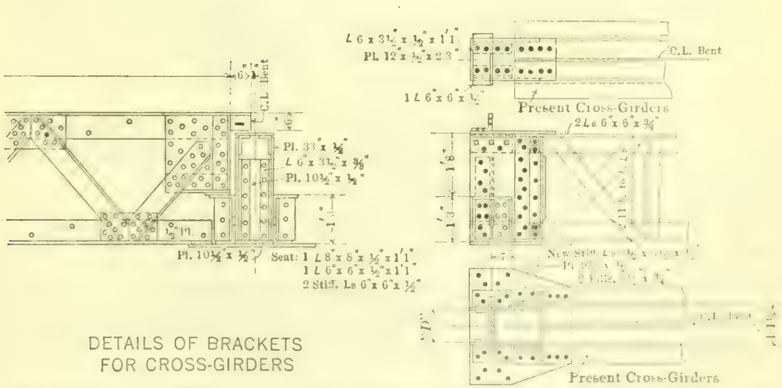


FIG. 79

and the other about 80 ft. long, which required reinforcing, in order to carry the load of the three tracks. The existing cross-bents were built up of Phœnix sections and tension bars, except the top flanges, which were shallow box-girders. The track structure in each span was carried by two trusses, which supported the track floor, consisting of cross-beams and secondary track stringers.

Fig. 80 shows the details of the reinforcement of the cross-trusses, which consisted mainly in relieving the bending stresses in the pins. The outside bars on each pin, which formed the center diagonals of the truss, were removed and replaced with plates to which new diag-

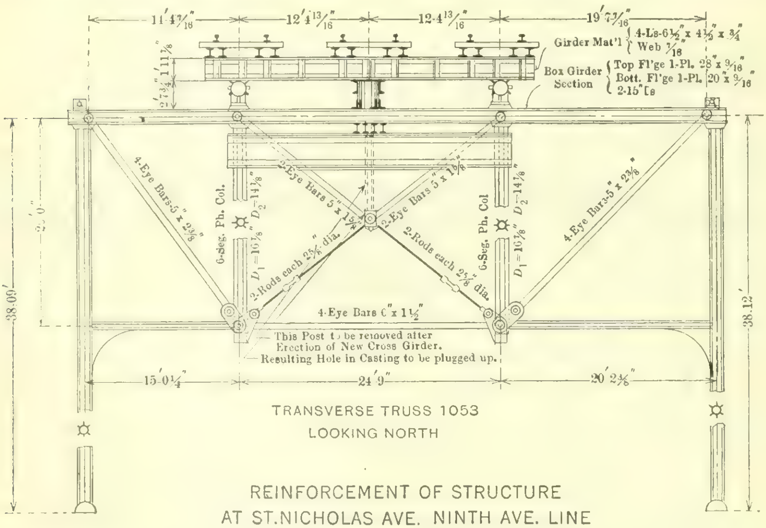
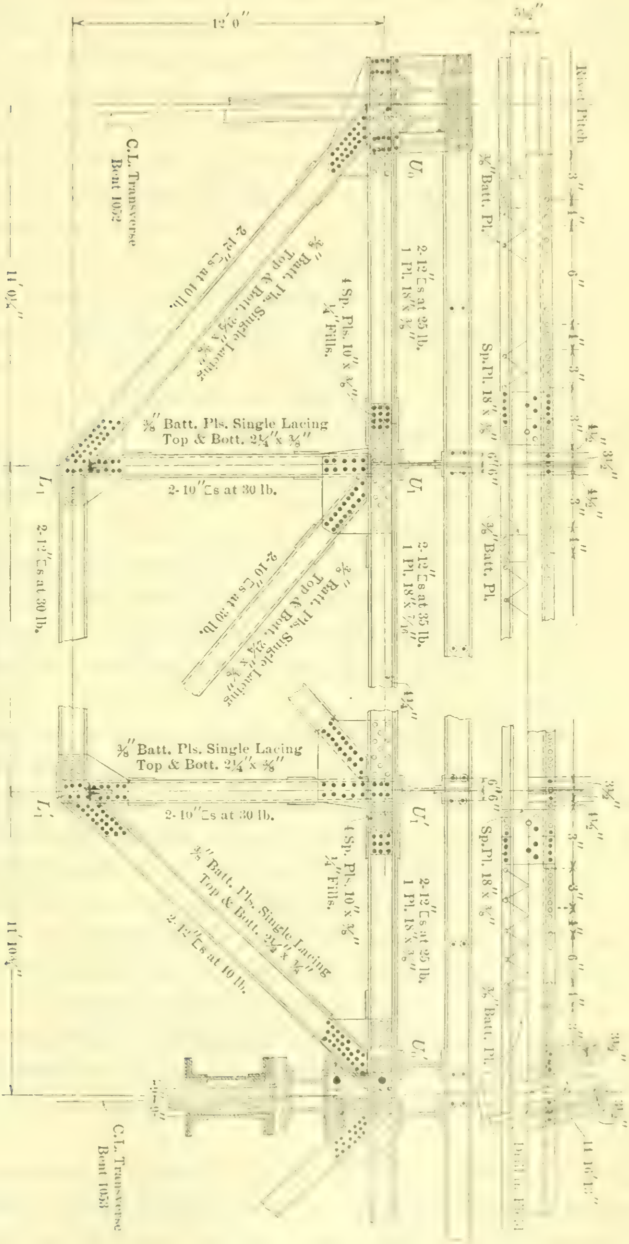


FIG. 80.

onals were fastened. To these plates were also riveted yokes directly under the pin which supported the existing truss members and thereby decreased the bending stresses on the pin.

The stresses in the longitudinal trusses were relieved by adding a center truss in both spans to carry the load of the center track. The details of one of these trusses, which had an expansion joint at one end, is shown by Fig. 81. The truss members were all made of latticed box sections, except the top chord, which had a top cover-plate. The top chord was placed at such a level that it supported the cross-floor beams. This brought the top flange below the box-girder, forming the top chord of the supporting cross-truss at the fixed end, and the



NEW TRUSS AT ST. NICHOLAS AVENUE
NINTH AVENUE LINE

FIG. 81.

connection to the cross-truss, therefore, was made at this end by a riveted yoke carried on the top of the cross-truss. At the expansion end the top chord of the longitudinal truss came above the cross-truss, and the support was made by an expansion rocker. In order to strengthen the top chord of the cross-truss at this point, a twin girder, as shown by Fig. 80, carried on brackets, was placed under the top chord and supported it by I-beams placed between the girder and the top chord. The brackets carrying the twin girders were riveted to the vertical posts of the cross-truss.

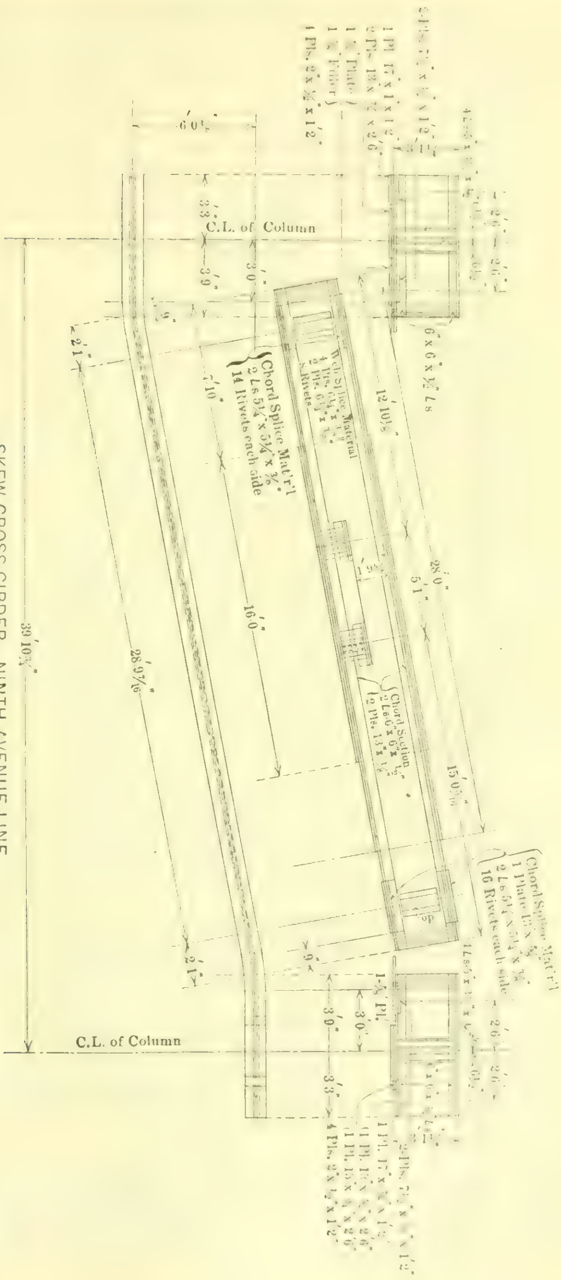
Steel Design.—Section No. 8-A.—The Ninth Avenue Line from south of the Cortlandt Street Station to near 12th Street was included in Section 8-A. The work embraced the addition of a center track throughout the section, and the reconstruction of the stations at Cortlandt, Warren, Desbrosses, and Christopher Streets.

The existing structure at the stations was so low that it was not possible to place mezzanines under it, but, on the other hand, it was wide enough to accommodate both the additional track and an express platform. The new platforms, therefore, were designed as island platforms and placed between the uptown local track and the new express track. Access to these platforms was obtained by an overhead bridge at each station, connecting to the express platforms, as well as both local platforms.

The existing structure was carried on two rows of fan-top columns on the sidewalk, each row carrying a single track. At only a few places the columns were connected with cross-girders. In order to place the center-track stringers, it was necessary, therefore, to place cross-girders, where none existed previous to the reconstruction work, remodel several of the existing ones, and reinforce the column tops in a manner similar to that described under Section 3. In addition, the columns supporting the structure at the reconstructed stations were replaced with new ones to carry the added loading.

Fig. 82 shows details of the new cross-girders. These were made in three pieces, spliced together. When the bents were skew, the cross-girders were curved at the ends to fit in between the existing track girders.

Details of Design.—Sections Nos. 8-B and 8-C.—Section No. 8-B comprised the reconstruction of the structure in Ninth Avenue to provide express stations of the "hump type" at 14th and 34th Streets; and



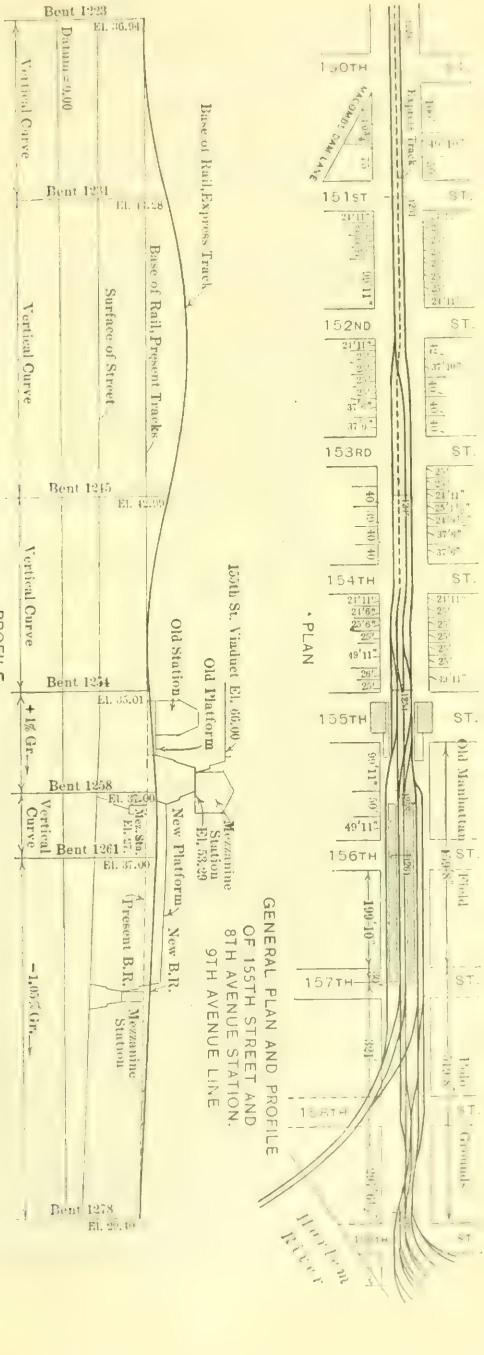
SKEW CROSS-GIRDER. NINTH AVENUE LINE

FIG. 82.

Section No. 8-C comprised the reconstruction required to eliminate the express track grade crossing at 53d Street, where the Sixth Avenue Line makes a junction with the local tracks of the Ninth Avenue Line. The details of the construction were similar to those described for other hump stations. The express track at 53d Street was carried high enough to obtain the proper head-room for crossing the track of the Sixth Avenue Line. The span length at the point of crossing had to be increased to 99 ft., and the structure across this span was designed as a through bridge construction.

Details of Design—Section No. 10-B.—The work on Section No. 10-B involved primarily the reconstruction of the station on the Ninth Avenue Line at 155th Street and Eighth Avenue to provide additional platform space for the express track and for the connection to the new elevated structure in Jerome Avenue. Fig. 83 shows the general arrangement of the station and its approaches when completed. The north-bound local track was divided into two tracks at 152d Street, and one of these was carried west of the express track. In order to avoid a grade crossing, the express track was carried above the grade of the local tracks. Between 152d and 155th Streets, where the rebuilt structure carried four tracks, the cross-girders had to be lengthened. The existing structure was similar to that described for the station at 125th Street, of the Ninth Avenue Line, under Section No. 7, and the details of the cross-girder extensions were the same. The details of the new overhead structure carrying the center track were also similar to those described under other sections, except the portion at 152d Street, where the local track crosses under the express track structure. Fig. 71 shows the details of this portion of the new structure. As it was necessary to carry the upper-grade track as low as possible, on account of the short length of the northerly approach, the track stringers of that track at the crossing were made very shallow. This was accomplished by cantilevering the stringers of the adjacent spans and supporting shallow track stringers on the cantilevers. The floor system of the upper-deck structure at the crossing consisted of floor-beams, framed into the cantilever girders, and of secondary track stringers carrying the track.

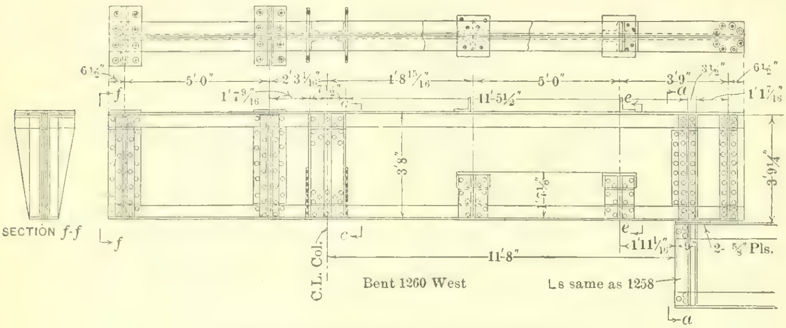
North of 155th Street the structure was widened by cross-girder extensions and new sidewalk columns, in order to carry the additional westerly tracks. Fig. 84 shows the details of these cross-girder exten-



PROFILE
FIG. 83.

GENERAL PLAN AND PROFILE OF 155TH STREET AND 8TH AVENUE STATION. 9TH AVENUE LINE.

sions. The existing cross-girders were twin girders; the extensions were made single-plate girders, set on top of the old cross-girders. The track stringers for the track nearest the old cross-girders were new, and were supported on seats. The outside track stringers were old, and of the 7-in. lip type. The lips of these girders were supported on top of wide seats provided for them on the cross-girder extensions.



DETAILS OF CROSS-GIRDER EXTENSION
FIG. 84.

FOUNDATIONS.

The foundation work involved not only the construction of foundations for new columns at points where no columns existed prior to the reconstruction, but also the rebuilding of the existing foundations where investigation indicated that these were not sufficient to carry the added loading. Altogether, 638 new column foundations were constructed, 444 of which were at new and 194 at existing locations. The placing of the foundations at new locations very frequently necessitated changes of gas and water pipes, duct lines, sewers, and other subsurface structures, because the locations of the columns were determined by other factors and practically without considering the subsurface structures, even if their positions were known in advance. The new column locations were determined in relation to the street traffic. Longitudinally, they had to be kept uniformly in line whenever possible and, in accordance with the Public Service Commission's certificate, they had to be at least 7 ft. from the center of a street-car track and 14 ft. from the curb, if they were in the street. At street crossings, they were placed so as not to interfere with the footwalk crossings; if on the sidewalk, their position was determined a definite dis-

tance from the curb. This distance was generally 8 in. from the edge of the curb to the face of the column, in order to be able to carry the curb stone through past the column, and also to prevent interference between the face of the column and the hub of a truck driving close to the curb.

If sub-surface structures were encountered, an investigation was always made first to determine whether the obstructions could be avoided by changing the shape of the foundation, and, where possible, the change was made. In some cases, when the location of the obstruction was known definitely prior to the fabrication of the columns, the interference could be avoided by lengthening the column shaft, and thereby bringing the base and the foundation below the obstruction.

At times small sewer pipes and house drains were run through and embedded in the concrete foundation; in such cases, cast-iron pipe was substituted for vitrified pipe; generally, however, pipes were offset around the foundation if the interference was local. When the same sub-surface structure interfered with several foundations, it was relaid in a new position, in order to avoid a large number of offsets.

The foundations were built of concrete, and were usually rectangular in plan. The top was slightly larger than the base of the column resting on the foundation. The sides tapered downward to the top of a rectangular footing which had vertical sides varying from 1 to 2 ft. in depth, and a horizontal offset all around, varying from 6 to 12 in. This shape was decided on, as it requires the least excavation and can be cast in one operation, without changing forms. The bottom area of the foundation depended on the load on it and the carrying capacity of the soil, but was generally not less than 7 ft. square. Usually, the size was 9 or 10 ft. square. The standard depth of the foundations was 14 ft. below the street surface; this depth generally assured the foundation against undermining due to excavations for future cellars or sub-surface structures.

Where the old foundations supporting the elevated structure were rebuilt, the superstructure was carried by shoring during their removal and the construction of the new ones. As the shoring had to support the structure in the immediate vicinity of the column, while it was desired to transfer the pressure at the street level to points sufficiently far away from the foundation excavation to avoid the necessity of excessive timbering and the danger of settlement of

the shoring due to a cave-in, the load was transferred by the shoring to girders supported at the ends by blocking, laid on the street surface. The girders were long enough to insure that the load on them would not produce excessive horizontal pressures on the sheathing holding the sides of the excavation. In most cases the shoring girders were either 24-in., 120-lb., Bethlehem beams, or 48-in. plate girders from 40 to 45 ft. long. The 24-in. beams were preferred, when strong enough to carry the load, as they were more easily handled. The blocking at the ends consisted usually of old ties laid side by side and wedged up slightly above the street surface, which was not disturbed. After the blocking and the girders were set in place and leveled up, the space between the blocking and the street surface was filled with a lean liquid mortar of cement and sand. It was found that this mortar when set produced an excellent bearing, and could easily be removed from the street surface, when the work was completed. No settlement of the street surface was ever observed due to the pressure of the shoring.

When conditions permitted, the shoring consisted simply of the shoring girders and brackets riveted to the columns and supported by the girders. This method was used when the structure had sufficient transverse stiffness, but only if the columns were to be renewed, as it was not desired to drill the holes necessary for the connection of the brackets in the webs or flanges of permanent columns. The column load was taken off the old foundations and transferred to the shoring girders by driving wedges between the brackets and the girders. When these wedges were being driven, the first effect was to deflect the girders, without raising the column; as the driving continued, the column, which had previously been made free to move vertically, by loosening the nuts of the anchor-bolts, would be lifted clear of the foundation. The driving of the wedges would be continued until the column was raised about ½ in. above the foundation. No work of removal of the foundation would be commenced until the column had remained wedged up for about 24 hours, in order to test the shoring under traffic. The wedging operation was carried on between trains, in order to avoid the additional live load due to the trains passing over the columns.

In most cases the structure of the single-column type was shored by placing four-post towers, made up of 12 by 12-in. yellow pine timbers on top of the shoring girders, one tower at each side of the

column. The structure was raised off the old foundation by wedging between the top of the towers and the bottom of the track stringers, the column which was connected to the track stringers being raised with the stringers. Special types of shoring used under extraordinary conditions will be described later.

The sides of the excavations were supported by 2-in. close sheathing, held in place by rangers, which were generally made of old 6 by 8-in. track timber, scarfed at the ends. The sheathing was driven down as the excavation proceeded.

At first the old foundations to be replaced were removed by hand. This method, however, was slow and expensive, and, as soon as air supply could be had, pneumatic hand drills were used instead.

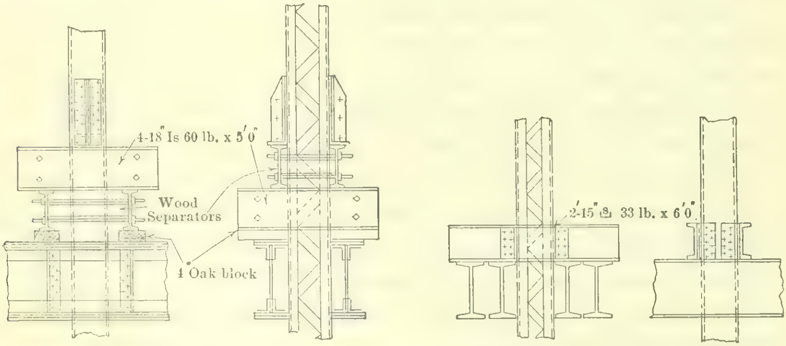
Usually, the sheathing was driven only to the top of the footing of the new foundations, and the concrete for this portion was placed without forms. Large stones were placed so as to project above the tops of the footings and form a bond with the remainder of the foundation. The forms for the tapered portion of the foundation were set on top of the concrete footing after the concrete had set. They were made in sections which could be bolted together and withdrawn without being destroyed; this allowed the use of the same forms for a large number of foundations. The framing consisted of 2 by 4-in. and 4 by 4-in. rough spruce, with planking of 2-in. boards, dressed on the edges and on the inside of the form.

The concrete used for the foundations was mixed in the proportion of 1 part of Portland cement, 2½ parts of sand, and 5 parts of stone graded to a maximum size of 2 in. On account of the long distance between the different points where the foundations were placed, and the comparatively small quantity of concrete required at each point, the mixing was usually done by hand. The mixture was made rather wet and was shoveled directly from the mixing board into the form. A laborer was generally stationed inside the form to distribute the concrete and work it well up against the sides.

The greater part of the foundation work occurred on Sections Nos. 1, 2-A, 2-B, 5-A, 5-B, 5-D, and 6-A, and only this work will be described herein.

Foundations.—Section No. 1.—Altogether, 64 foundations were constructed in this section, of which 52 replaced existing ones, and 12 were in new locations. In several places, where shoring was needed,

the gas mains were found to be so close to the street surface that provision had to be made against danger to the shoring from possible explosions and fire due to leaks in the mains, which danger often was aggravated by the necessity of placing the support of the shoring directly over the gas mains. This danger was provided against by drilling and plugging holes in the mains about 250 ft. apart. If a leak occurred, the flow of gas was stopped by bagging the adjacent holes until repairs were made, and if the leak was directly under the shoring, where no repairs could be made during the time the shoring carried the load, the pipe was by-passed around the shoring, and the interrupted house service connections were connected temporarily to the by-pass.



SHORING BRACKET NO. 2 — SHORING BRACKETS NOS. 3, 4, 5 AND 6

Fig. 85.

In practically all cases the load of the structure was transferred to the shoring girders by brackets riveted to the column shaft, as shown by Fig. 85. When the load was less than 200 000 lb., the bracket consisted of two 15-in. channels connected to the webs of the column channels by four angles. These 15-in. channels extended far enough on both sides of the column to bear on the supporting shoring girders. When the load was greater, the brackets consisted of inverted seats riveted to the webs of the column shaft and supported on two 18-in. or 20-in. I-beams, which, again, were supported on I-beams laid crosswise so as to get a bearing on the shoring girders. Fig. 87 shows the structure in the New Bowery supported on brackets of the first type. Here the shoring girders of some of the bents were carried several feet above the sidewalk level, in order not to interfere with the surface structures, such as fire hydrants and coal holes.

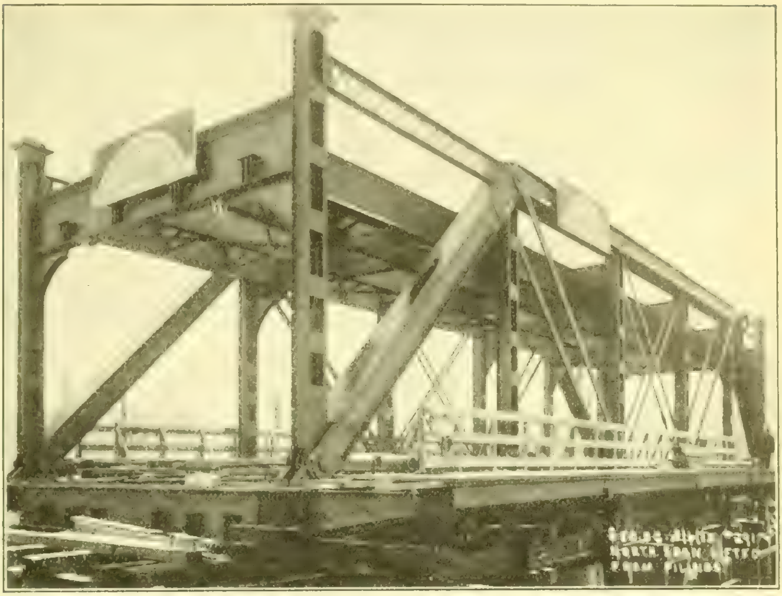


FIG. 86.—NEW NORTH SPAN OF HARLEM RIVER BRIDGE.

FIG. 87.—SHORING FOR FOUNDATIONS. NEW BOWERY.

A special method of shoring was used for a column at Chatham Square. This column was so close to a surface-car track, that there was not sufficient room to place a shoring girder on the side of the column adjacent to the car track.

The shoring as shown by Fig. 88 was accomplished by placing on the street surface two plate girders, 4 ft. deep, on the side of the

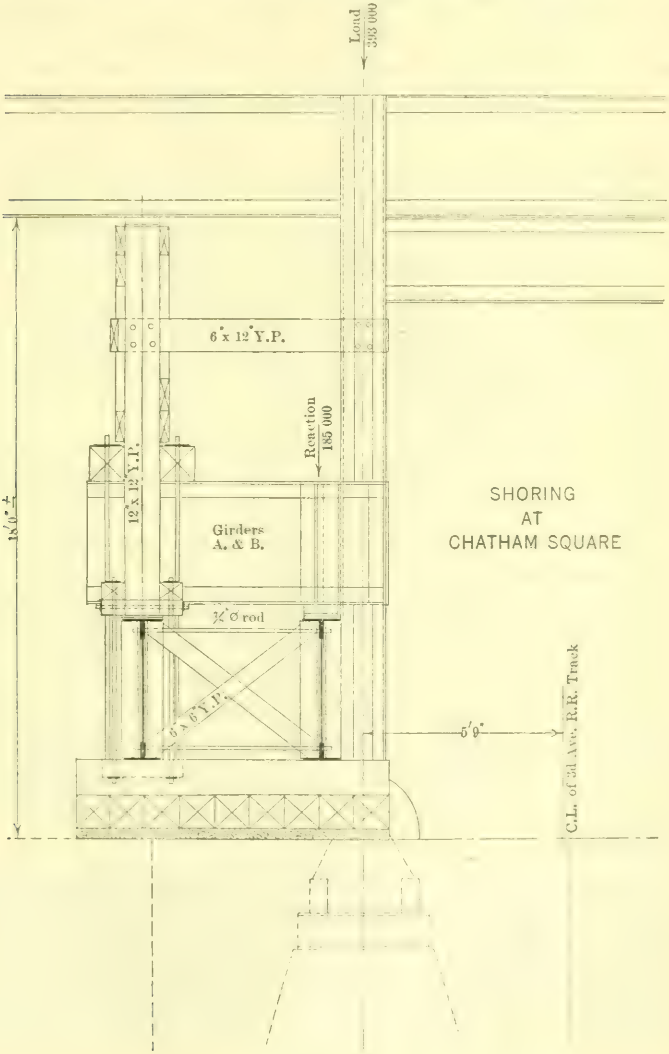


FIG. 88.

column away from the street-car tracks. The girders were placed approximately parallel to the car tracks, one close to the column and the other about 5 ft. from the first. By using timber bents, the load of two of the cross-girders carried by the column was brought to bear on the top of the second shoring girder. The remainder of the load on the column was carried by two short cantilever girders placed on top of, and at right angles to, the shoring girders and riveted to the column. The shoring girder adjacent to the column acted as a fulcrum for these cantilever girders, and the uplift at the outer end of the cantilevers was taken care of by fastening them with yokes to the outer shoring girder. All connections of the short cantilever girders were made to the webs of the column channels, as it was not considered safe to remove the lattice bars connecting the channels, in order to rivet the cantilevers to the flanges of these channels, because the column was taking its full load while the cantilever girders were being riveted. A considerable portion of the load on the column was taken directly into the shoring girder adjacent to the column, due to the fact that two of the angles connecting the column to the cantilever girders were placed directly over the shoring girder.

As the new foundations were completed in advance of the delivery of the new steelwork, the old columns were supported temporarily on the new foundations. The bases of the old columns, however, were generally at a higher elevation than those of the new columns. The concrete foundations were finished at their proper level, but on top of the concrete was built a brick extension, high enough to support the old columns. Brick was used for this work, in order to facilitate its removal when the old columns were replaced with the new structure.

In general, all new column bases were designed to fit over the old anchor-bolts, or, where new anchor-bolts were used, the latter were dropped through the holes in the cast-iron bases of the old columns which took the old anchor-bolts, and were then built into the new concrete foundations.

In certain cases, however, the details of the new structure necessitated different locations for the anchor-bolts that secured the existing column and the new column to the new foundation. In such cases, two sets of anchor-bolts were provided, and the temporary ones were cut off flush with the top of the permanent foundation when the new column was erected. The anchor-bolts for the new columns, however, could

not extend during the temporary period above the bottom of the old base. Fig. 89 shows such a case. As the new column was skewed in relation to the existing column, two sets of anchor-bolts were provided. The anchor-bolts for the new structure had sleeve nuts, set flush with the top of the permanent foundations. When the old steelwork was removed, extensions of the anchor-bolts were screwed into the sleeve nuts, and the necessary length of the bolts was obtained in this manner.

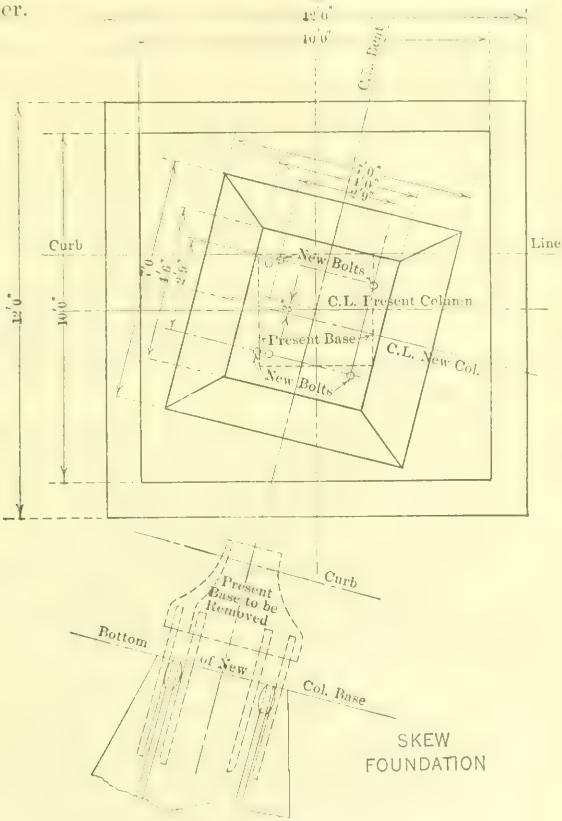


Fig. 89.

Foundations.—Section No. 2-A.—In the Bowery, for the portion of the Third Avenue Line included under Section No. 2-A, 66 new foundations were placed. Of these, 32 were in the roadway, 17 on the sidewalk clear of the existing foundations, and 17 replaced existing foundations.

The Bowery is an old established main thoroughfare, and contains a vast number of sub-surface structures, the location of which it was desired to determine in advance, so that they could be removed where they interfered with the construction of the foundations. Trenches were excavated across the street, therefore, first on one side, between curb and car track, and then on the other side, so as to maintain the street traffic. These trenches were made on line with the location of each new foundation, and were about 2 ft. wide and about 5 ft. deep, which is generally as deep as any sub-surface structures, except sewers, will be found. All the sub-surface structures encountered were surveyed and plotted, and the necessary relocations determined. Between Canal and Hester Streets, the relocation involved, on the west side of the street, a 12-in. gas main, a 16-in. gas main, a 24-in., high-pressure (fire service) water main, and, on the east side, a 16-in. gas main, an 8-in. gas main, and a 12-in. water main.

After the sub-surface structures had been relocated, which was done in sections, and on one side of the street only at a time, the new foundations within the excavated area were constructed, and the whole excavation was refilled and repaved.

At the points where the foundations were replaced, the structure was supported by shoring girders. At some of these points the usual method of shoring could not be used, as the columns were near street crossings where the shoring girders, as usually arranged, would interfere with the traffic on the street-railway tracks. Therefore, the arrangement shown by Fig. 90 was used. An excavation, 10 by 8 ft. was made immediately adjacent to the foundation to be reconstructed, and carried down to the full depth of the new foundation, which was about 8 ft. below the existing one, and, therefore, necessitated careful sheathing. The bottom of the excavation was covered with concrete on which was laid a solid floor of 12 by 12-in. timbers. On the top of this floor was set a four-post timber bent with two of the posts vertical and the other two on a batter. The posts were 12 by 12-in. yellow pine timbers, and the bent was 19 ft. high. Near the column of the bent, next to that for which the foundation was to be reconstructed, another timber bent was placed, resting on the street and sidewalk surface. The shoring girders were placed on top of these bents and cantilevered 8½ ft. over the bent first described, so as to reach the

point of the structure to be supported. The uplift was taken care of by blocking the shoring girders against the track stringers.

Foundations—Section No. 2-B.—In this section, which extended on the Bowery from Delancey to 5th Street, practically all the columns were moved from the curb line to the roadway. The greater part of the work consisted of changes in the sub-surface structures. On account of the excessive number of such structures, it was not possible to provide room for all of them outside of the foundations, and one 16-in. gas main, therefore, was conducted through the center of the

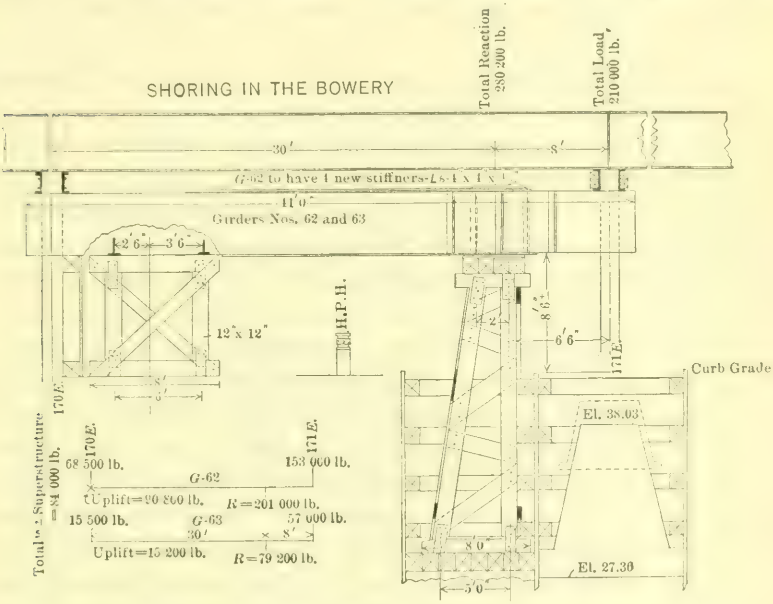


FIG. 90.

foundations at both sides of the street. This gas main, which consisted of cast-iron pipe, was first jacked into such a position that it would be centered in the foundations throughout the whole length, and those portions of the main which would be within the foundations were then replaced with wrought-iron pipe. In order to make it possible to withdraw and renew the pipe, a form was placed around it prior to replacing the concrete. The form was made of a steel plate, semicircular at the top, and with straight sides. The bottom of the form was a 2-in. plank. The steel form was made in two lengths, meeting at the center of the pier, and was held in place by wooden

blocks, 1 in. thick, placed on top of the pipe. The form was removed by pushing these blocks beyond the form and allowing it to drop. The wrought-iron pipes were placed in such a manner that no joints occurred inside of the foundation, and the minimum height of the concrete above the pipe was 1 ft. The column bases supported by the foundations have been described previously.

Foundations.—Section No. 5-A.—The work in this section comprised the rebuilding of 26 foundations under the existing structure and the placing of 9 additional foundations, 8 of which were on the sidewalk and supported the added platform structure of the station at 125th Street.

At the time the work was done, only 24-in. Bethlehem I-beams were available for shoring girders, and three of these were needed to carry the load of one column. One 24-in. beam was placed on the side of the column adjacent to the street-car tracks, which were in the center of the street between the columns supporting the structure. There was only room for one I-beam on this side of the column between it and the street-car clearance, otherwise four I-beams would have been used, in order to avoid eccentric loading. The details of the shoring are shown by Fig. 91. Brackets riveted to the column shaft, and consisting of two 15-in. channels, transferred the load to the shoring girders. The deflection of the 24-in. I-beams was sufficiently large to gauge the uniformity of the stress in the three I-beams by the uniformity of their deflection, which was maintained by driving the wedges the proper amount on top of each I-beam. In order to avoid undue stress in the long arms of the brackets, a timber bent tied to the column shaft was set on top of the brackets and was supported by the under side of the existing track stringers, and wedges were driven between the top of the bent and the track stringers in order to prevent undue deflection of the brackets.

Foundations.—Section No. 5-B.—On this section no column foundations were removed and replaced, but 27 new foundations were built, of which 24 were built on the sidewalk to support the new 125th Street Station, and were of the standard type; the remaining three were of a special type, and deserve further description. These three foundations were in close proximity to the west shore line of the Harlem River, at the point where the bridge carrying the Second and Third Avenue Lines crosses the river. The surface of the ground is

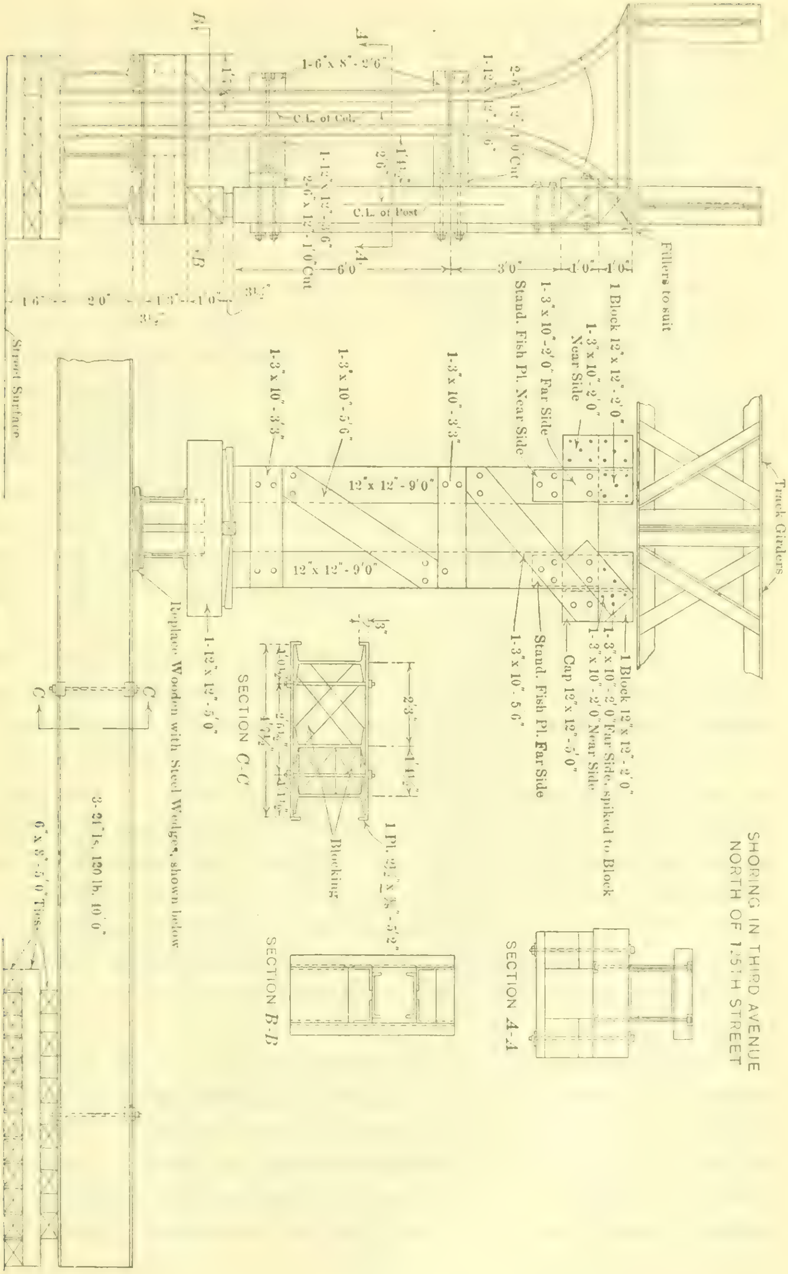
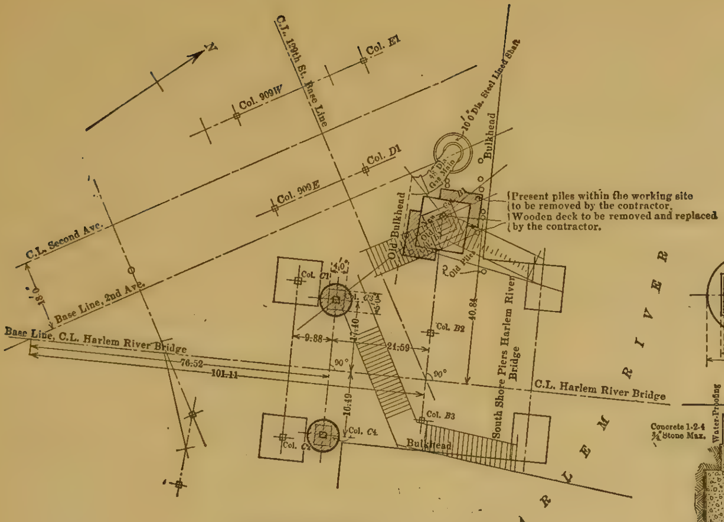

SHORING IN THIRD AVENUE
NORTH OF 125TH STREET
SECTION A-A
SECTION B-B
SECTION C-C
Track Girders
Fillers to suit
C.L. of Col.
C.L. of Post
Street Surface
12"x 12"- 9'0"
12"x 12"- 9'0"
Cap 12"x 12"- 5'0"
Stand. Fish Pl. Near Side
Stand. Fish Pl. Far Side
Replace Wooden with Steel Wedges, shown below
3-24" Is. 120 lb. 10'0"
6"x 8"- 5'0" Ties
Blocking

FIG. 91.

less than 6 ft. above mean high water, and the soil is very soft to a depth of more than 30 ft. below mean high water, where good coarse sand is found. The south shore piers of the Harlem River Bridge and a number of foundations carrying the elevated structure are in this vicinity. The bridge piers are carried down solidly to firm bearing; the structure foundations consist of masonry piers to mean low water, supported on piles. The softness of the soil causes it to have a tendency to move toward the river, as shown by the existing foundations, the tops of which, in the course of time, have shifted an appreciable distance, making it necessary to disconnect the columns from the foundations and reset them. Although the existing foundations might have been sufficient to carry the additional loading of the new structure, it was not deemed advisable to add to their loading. The new structure, therefore, was supported on new and independent foundations. It was desired, however, to construct these new foundations in such a manner that they would be able to resist the tendency to move toward the river, and so that, at the same time, their construction would not disturb the existing structure or bridge foundations.

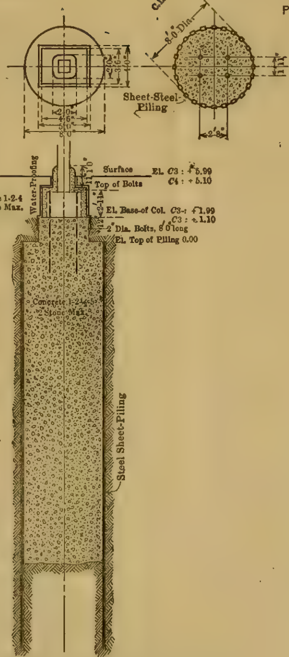
The details of the designs and the location of these three foundations are shown on Plate XLVII. All three foundations consisted of concrete carried down to solid ground and encased in a steel sheet-piling. Two of them were circular. The third was intended to replace an existing foundation without removing the latter. It was built, therefore, around the existing foundation, but on account of various sub-surface obstructions, it had to be divided into two parts. The obstructions consisted of a 48-in. gas main curving around and into a steel-lined shaft, 10 ft. in diameter, from which it was carried in a tunnel across the river, an old cribwork bulkhead, a pilework bulkhead, and six submarine cables, supplying electric current for the Borough of The Bronx.

The construction of these foundations was done in the following manner: The steel piling for a foundation was first driven to the full depth. The excavation was then made, without attempting to remove the water, in fact, it was desired to retain the water in order to lessen the pressure on the piling and decrease the possibility of material flowing from the outside into the excavation. Finally, the concrete was placed, through the water.

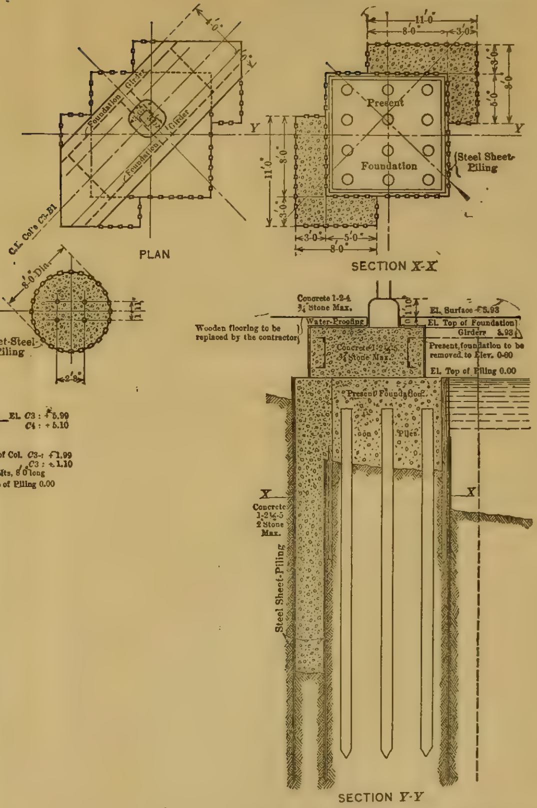


LOCATION DIAGRAM

FOUNDATIONS, SOUTH OF HARLEM RIVER BRIDGE
SECOND AVENUE LINE



DETAILS OF FOUNDATIONS FOR COLUMNS $C3$ AND $C4$



DETAILS OF FOUNDATIONS FOR NEW COLUMN "B"

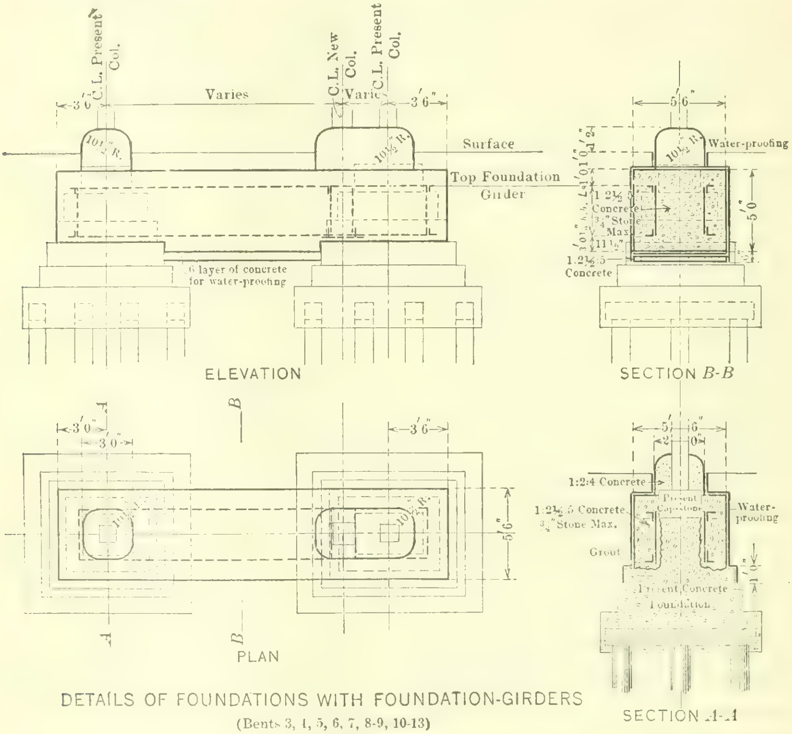
One of the cylindrical foundations was first started. On account of the foundation being partly under the existing structure, the sheet-piling could only be set in short lengths, and three lengths were used to bring it to the full depth. A pit was first excavated by hand to low-water level and shored. A wooden drum, about 8 ft. in diameter, was then set in the pit as a guide for the sheet-pile driving, and the first length of sheet-piling was set up and interlocked complete. Lackawanna steel sheet-piling, $12\frac{3}{4}$ by $\frac{1}{2}$ in., was used. The lengths of the pilings were 10 ft. and 14 ft., alternately, so as to form a splice 4 ft. long for the next section. The driving was done with a 2-ton steam hammer. Each piece of piling was hammered down about 1 ft. at a time, around the complete circle, a follower, consisting of a 4-ft. length of sheet-piling, being used to drive the 10-ft. sections. The piling drove easily in the soft ground, but rip-rap and heavy logs, probably part of former cribwork construction, retarded the progress of the work. The logs lying horizontally formed in particular very obstinate obstructions, as they acted as an elastic cushion and caused the piling to bounce back after each blow without any apparent effect on the log. Sometimes it was necessary to pull some of the pile sections, sharpen the edge of the interlocking part of the pile, and start driving again, and finally the log would give way rather suddenly.

The excavation was done with a clam-shell bucket worked by a derrick, which had also been used to suspend the pile-driver and move it from place to place. No serious difficulties were met in doing this work. The concrete was placed through a chute, the water never being removed during the operations of excavation and concreting.

The second circular foundation was completed in a similar manner. The third foundation was started by surrounding the existing pier completely with steel sheet-piling, in order to tie the two parts of the new foundation together. The remainder of the piling for the first length was then set, and driving commenced. Before driving had started, it became apparent that the submarine cables previously mentioned might possibly have shifted from their original position and might be directly under the sheet-piling to be driven. A diver, therefore, was employed to locate the cables and remove them, if necessary. As some of the cables had been placed prior to the shaft sinking for the gas main, they were buried from 8 to 10 ft. below earth and rip-rap. The rip-rap was removed by the diver, who placed it in a bucket which

was hoisted from the top, the soft soil was removed by a water jet, and the cables were replaced so as not to interfere with the driving of the sheeting.

The piling was driven until hard bottom was encountered, which was 34 ft. below mean high water on the land side and 44 ft. on the river side. This difference was probably due to the disturbance of the soil on the river side during the shaft sinking for the gas main.



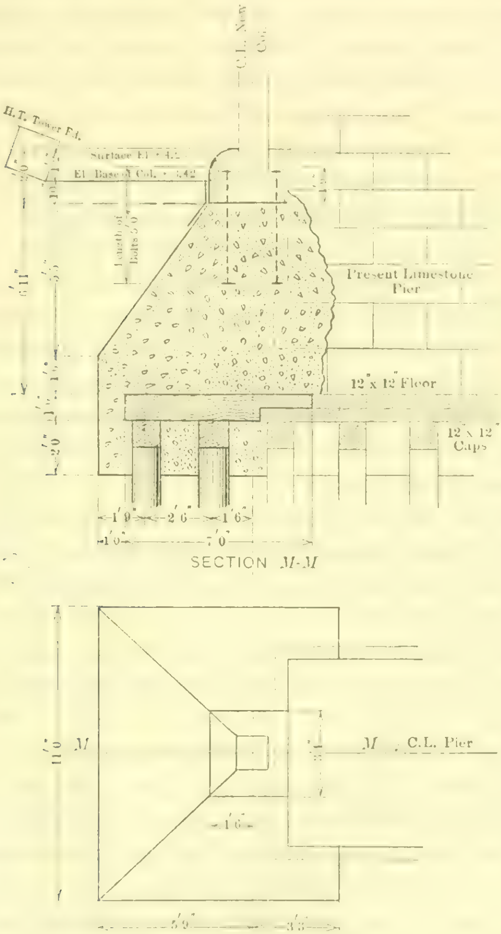
DETAILS OF FOUNDATIONS WITH FOUNDATION-GIRDERS

(Bents 3, 4, 5, 6, 7, 8-9, 10-13)

FIG. 92.

Foundations.—Section No. 5-D.—The portion of the existing structure which was within the limits of the freight yard of the New York, New Haven and Hartford Railroad Company was generally supported on four-column bents placed between and parallel to the yard tracks. Each column was carried on a separate concrete foundation with a granite cap, and supported on piles. The new columns were on a line with the existing bents and between a pair of existing columns, but generally so close to one of them that a new foundation

could not be placed without interfering with the existing one. In addition to this, the driving of piles for new foundations was out of the question on account of the interference with the traffic in the railroad yard and the lack of head-room under the existing structure.



SECTION M-M

PLAN

DETAILS OF NEW FOUNDATIONS
NEAR HARLEM RIVER

FIG. 93.

The method used for supporting the new columns, therefore, was to bridge the space between the existing foundations with two foundation girders supported by the existing foundations, and to connect the column to these girders. Fig. 92 shows the details of these founda-

tions. Each of the foundation girders, which were 3 ft. deep, consisted of a web-plate and a single top and bottom flange angle. They were spaced generally 3 ft. apart, which brought them outside of the cap-stone of the existing foundations. The concrete of the existing foundations was trimmed down, so as to make room and provide a seat for the foundation girders. After the girders had been set in place, the spaces between the concrete pier and the foundation girders were filled with grout. Diaphragms were provided at the ends of the foundation girders to keep them in place and at the location of the columns to carry their loads. When the riveting was completed, the foundation girders were encased completely in concrete which was water-proofed all around with four-ply water-proofing paper and pitch.

The details of the foundations for the new columns of the first two bents of the Harlem River are shown by Fig. 93. As neither railroad tracks nor existing structure interfered with the pile-driving at these locations, additions were made to the existing foundations, which were supported on piles. The additions were made to correspond to the construction of the existing foundations so as to prevent, as far as possible, uneven settlement.

North of the railroad yard tracks, where the new columns came close to the existing foundations, similar additions were made to the latter with the exception, however, that no piles were used to support these foundations.

Altogether, in this section, 14 new columns were supported on girders, 12 on additions to existing foundations, and 21 on independent foundations.

Foundations.—Section No. 6-A.—This section was entirely on a private right of way, except where the right of way intersected cross streets. The existing structure was supported directly on brick piers throughout the private right of way, and on column bents set inside the curb lines at the cross streets. As the existing south-bound track was moved sidewise, new piers had to be constructed to support this track, in addition to the new foundations required to support the additional structure. Altogether, 53 new piers and 205 new foundations were constructed in this section.

The existing bents at the cross streets were supported on four columns set on the sidewalk parallel to the curb. The new supporting columns of the bents were on line with the existing columns, but gen-

erally at different locations. Plate XLVIII shows the method of shoring used on rebuilding the foundations. First, four shoring girders, consisting of 24-in. Bethlehem, I-beams, or two plate girders, 4 ft. deep, were placed parallel to the bent, two on each side of the columns, and approximately centered on one of the existing tracks. These shoring girders were supported on timber blocking at the points marked *A*, *B*, and *D* (Sketch No. 1). The cross-girder carrying the existing track was then supported on blocking between the top of the shoring girders and the bottom of the cross-girder, and the load was transferred to the shoring girders by driving wedges between the blocking and the cross-girder. The existing foundation under the outside column was then removed, and the new one (marked "1") was placed. At the same time, the other foundation (marked "2"), which did not interfere with the existing structure, was constructed. After back-filling around Foundation 1, so that the blocking (marked *C*), under the shoring girders, could be placed; this was done and the blocking (marked *B*) was removed. The existing foundation for the inside column of the supported track was then removed, and the new foundation was placed. As the existing column had to be supported until the new steel could be erected, the new foundation was made wide enough to carry this column. A brick support was built between the top of the permanent foundation and the under side of the existing granite cap-stone, which was attached with bolts to the column and retained, in order to facilitate the removal of the temporary support. When this work was completed, the shoring girders were moved so as to center on the other track, and were supported at one end on the new foundation (marked "2") and at the other end by a framed timber bent set on the base of the newly constructed foundation (marked "3"). The track structure was supported by blocking, as before. The foundations under the two shored columns were then removed, and the new permanent foundations were placed. This new foundation was made wide enough to support the existing columns temporarily.

At 138th Street, where a new subway was being built, it was desired to carry the foundation down to solid rock. The rock surface at this point was very irregular, so that, though two of the foundations were carried only 17 ft. below the surface of the street, three of them had to be carried down about 39 ft., which brought them below the subgrade of the subway.

Fig. 94 is typical of the foundation work within the right of way. The light lines represent the piers supporting the existing structure, and the heavy lines represent the new work. The new pier was built to support the existing south-bound track, which was moved sidewise, and the foundations supported the new structure. The new piers were generally carried down to the bottom of the existing piers, about 6 ft. below the surface, but, in addition to this, before the piers were built, an excavation was made along the property line under each pier, 3 ft. wide, 16 ft. long, and about 12 ft. deep below the curb level of the nearest street, and filled with concrete, in order to prevent possible undermining of the piers, if the adjoining houses should be rebuilt.

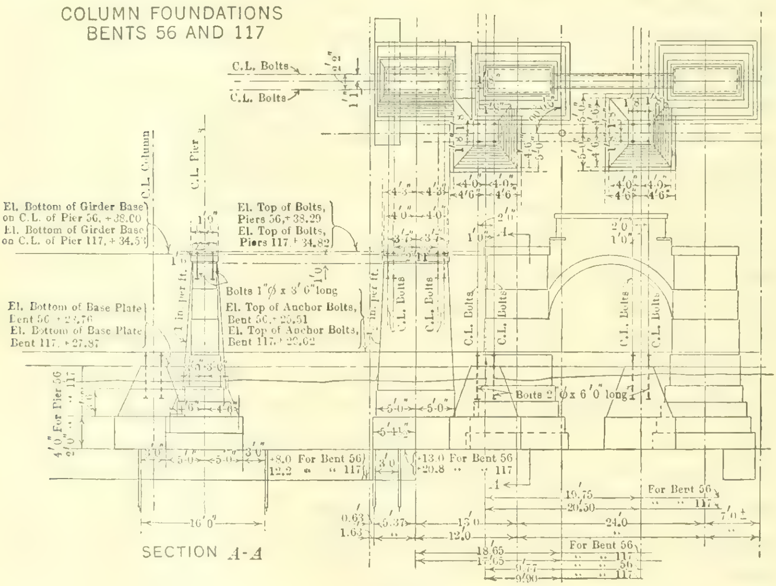
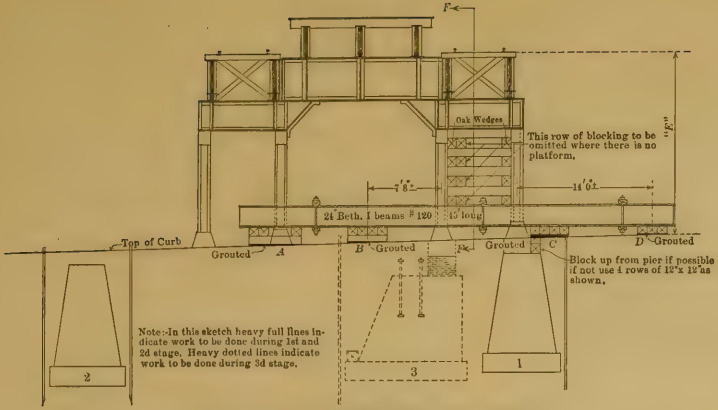


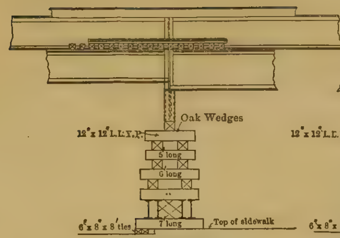
FIG. 94.

The foundation and the piers on the west side of the right of way were completed first, starting from the south end and going north. The remaining foundations on the east side were then constructed, starting at the center of a block and working in both directions in order to facilitate the handling of the materials in the limited working space.

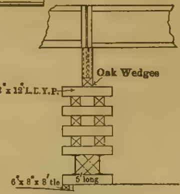
The forms for the piers were detachable, and each side was in three sections and secured in place by timber yokes. On account



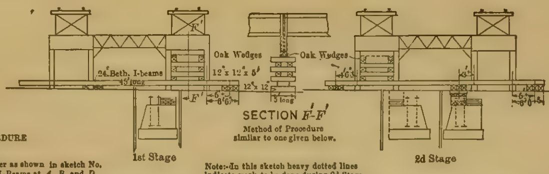
SKETCH NO. 1.—1st 2d AND 3d STAGE, LOOKING NORTH.



SECTION F-F
Where there is a platform.
Bents 59-60-80-81-115-& 116.



SECTION F-F
Where there is no platform.



SKETCH NO. 3.—TYPICAL SHORING FOR BENTS 129-130 AND 136, LOOKING NORTH.

METHOD OF PROCEDURE

1st Stage

Shore up North-Bound Cross-Girder as shown in sketch No. 1, placing the blocking under the 24" I-Beams at *A*, *B*, and *D*. Excavate for and Build foundation No. 1.

2d Stage

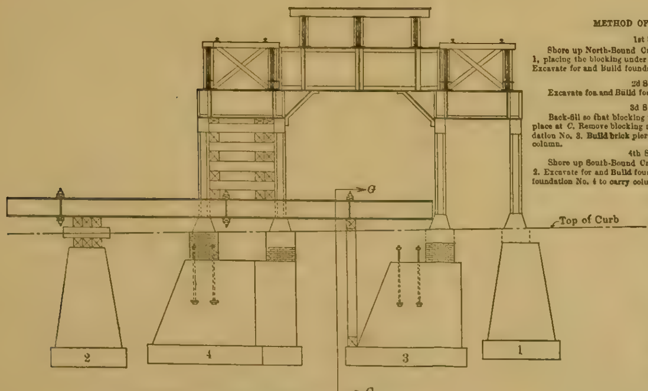
Excavate for and Build foundation No. 2.

3d Stage

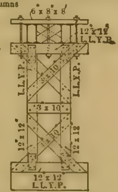
Back-fill so that blocking under 24" I-Beams can be put in place at *C*. Remove blocking at *B*. Excavate for and Build foundation No. 3. Build brick pier on foundation No. 3 to carry column.

4th Stage

Shore up South-Bound Cross-Girder as shown in sketch No. 2. Excavate for and Build foundation No. 4. Build brick pier on foundation No. 4 to carry columns



SKETCH NO. 2.—FINAL STAGE, LOOKING NORTH.



SECTION G-G

SHORING FOR FOUNDATIONS
IN STREETS
BETWEEN 134TH AND 145TH STREETS

of the large quantity of concrete to be mixed, a concrete mixer was used for the construction of the piers. The mixer was mounted on a platform which also carried the implement for hoisting and dumping the concrete. The platform was moved on rollers, using a drum on the hoist for motive power.

Steel Erection.

The greater part of the work under the Manhattan Elevated Railway Improvements was in public streets, often congested with traffic. The steelwork, the total weight of which was about 50 000 tons, therefore, could not be stored in large quantities, or for any length of time, at the points where it was required for erection. The greater part of the steel came over the Pennsylvania Railroad and the Central Railroad of New Jersey, and was unloaded and stored in the railroad terminal yards in New Jersey, at Greenville and Communipaw, respectively. The receiving points for the steel in New York were the Company's Yard at 128th Street and the Harlem River, the dock front of which was increased by leasing adjacent property, and a dock front at Perry Street and the North River was leased for the purpose. All the steelwork for the up-town sections was delivered at the 128th Street dock, and that required for the down-town sections at the Perry Street dock. As the material was needed, it was lightered from the railroad yards to the Company's docks, where it was stored temporarily until it could be trucked away. The trucks were loaded at each dock with a derrick erected for the purpose. The material was trucked to the site of the work and deposited in the street until it could be erected, which generally meant that it remained in the street only for a day or two. In this manner, the congestion of the streets due to storage of material was reduced to a minimum.

Steel Erection.—Section No. 1.—Fig. 95 is a bird's-eye view of the greater portion of Section No. 1 after completion. The view was taken from Chatham Square, and shows the City Hall Branch in Park Row to the right and the South Ferry Branch in the New Bowery to the left. Fig. 96 shows the general layout of the old tracks at Chatham Square. The first work done was the erection of the steel for the west platform of the new City Hall Branch Station at the west side of Chatham Square. The steelwork for this platform and the adjacent tracks is shown on Fig. 96. When the station was

completed, and the new west track was connected to the existing south-bound track, the south-bound service was diverted from the existing station just north of Chatham Square to this new station and track. The old south-bound track was then removed, so that connection could be made to the new north-bound track east of the new platform. When this work was completed, the old station was abandoned and the structure carrying the disused portion of the tracks was dismantled, making room for the construction of the new two-track structure, connecting to the new upper deck of the City Hall Station.

The erection of the new structure in the New Bowery was started at the south end, where the ramp commenced at the grade of the present structure. The erection of this structure involved only minor changes in the existing tracks. The new structure in Park Row was erected with a traveler, starting at Chatham Square and going through on the upper deck to City Hall Station. When the new structure had risen sufficiently above the lower deck, it was carried immediately above the existing tracks. Before the traveler erected the upper-deck girders, the new columns were erected, and the lower-deck cross-girders were either reinforced or replaced. Fig. 97 shows the shoring of the structure used to carry out this work. Two four-post bents were placed at each side of the structure and supported two shoring girders which carried the track stringers and the track while the old columns and cross-girders were being removed and the new ones placed. The erection of the new work was done with gin-poles, and, in order to facilitate their erection, the new lower-deck cross-girders were shipped bolted and, in the beginning, raised member by member and riveted in place. As the work progressed, it was found that they could be raised complete without serious interference with the traffic in the street, and, therefore, they were riveted together completely before erection with the exception of the top flange, which had to be erected separately, as it extended above the top of the track stringers, the ends of which were so close together that the flange of the cross-girder could not pass through the intervening space.

Steel Erection.—Section No. 2-A.—Whenever the erection of the new structure necessitated interference with the train service, the method of procedure was carefully worked out in advance, so as to insure at all times during the progress of the work safe and continuous

FIG. 95.—CHATHAM SQUARE, LOOKING SOUTH.

FIG. 96.—CHATHAM SQUARE AT THE BEGINNING OF RECONSTRUCTION.

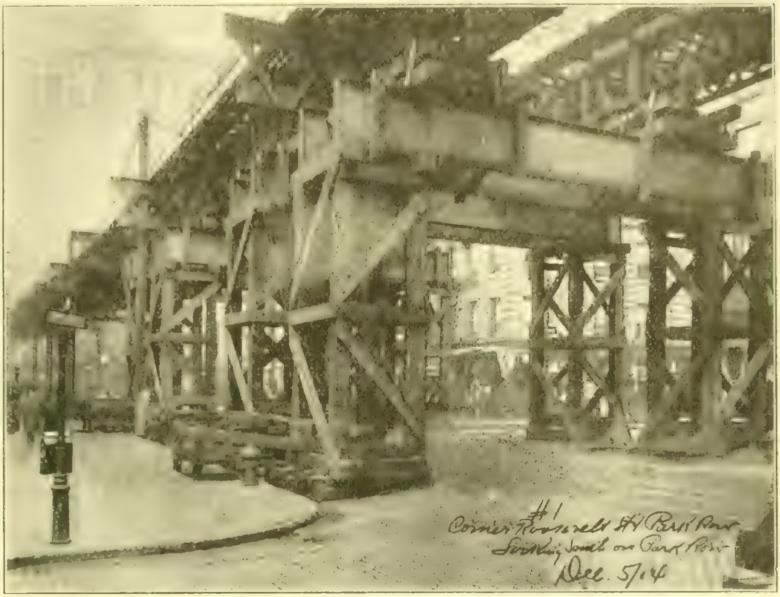


FIG. 97.—SHORING IN PARK ROW.

FIG. 98.—NEW AND OLD STRUCTURES IN THE BOWERY.

operation of trains and station facilities. Plate XLIX shows the method of procedure worked out in advance of, and strictly followed in the erection of, the new steel structure in Section No. 2-A. The plan marked "Original Condition" shows the platforms and tracks which existed prior to the reconstruction work. It shows the old Chatham Square Station, with the pocket tracks, the Canal Street Station, with

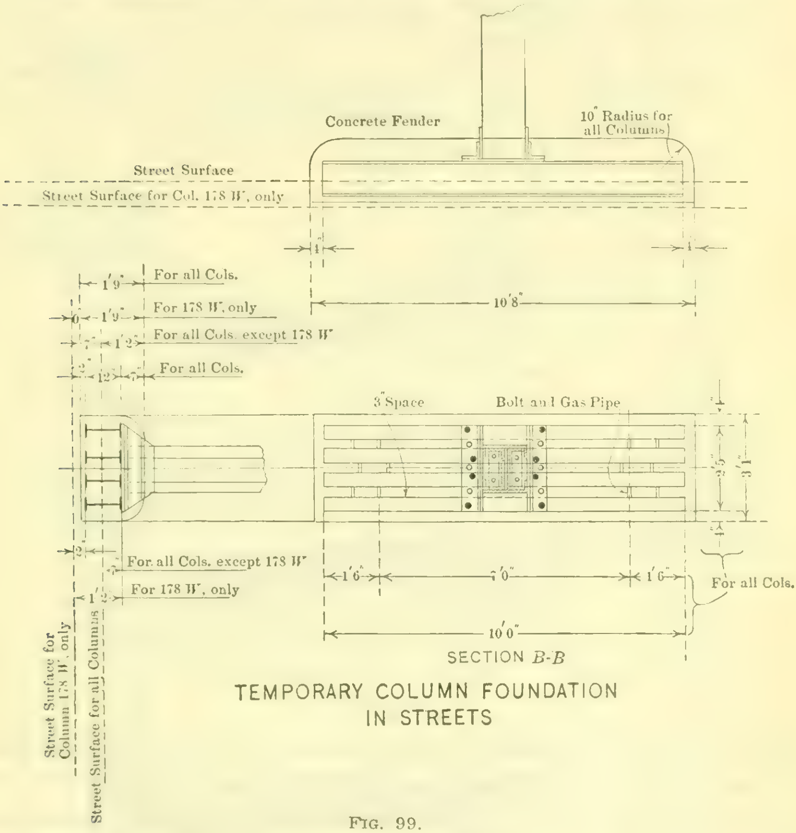


FIG. 99.

the outside platforms, and the Grand Street Station, with two inside platforms, to which access was obtained by stairs under the tracks. The new steelwork through the old Grand Street Station could not be placed until the new station had been erected, as the existing platforms and station building, which covered the entire space between the tracks, had to be maintained. The steel erection, therefore, was commenced north and south of the old Grand Street Station.

South of Grand Street Station, the two bents, numbered 191 and 192, were first erected with a derrick set up in the street. As the new permanent columns on the west side of the street were to be on the sidewalk under the existing structure, and, therefore, with the cross-girders supported by them, would interfere with the existing track structure, the new cross-girders were designed so that those portions which did not interfere with the existing track structure could be erected first. Temporary steel columns were provided to support the west ends of these girders, as shown by Fig. 99. The temporary

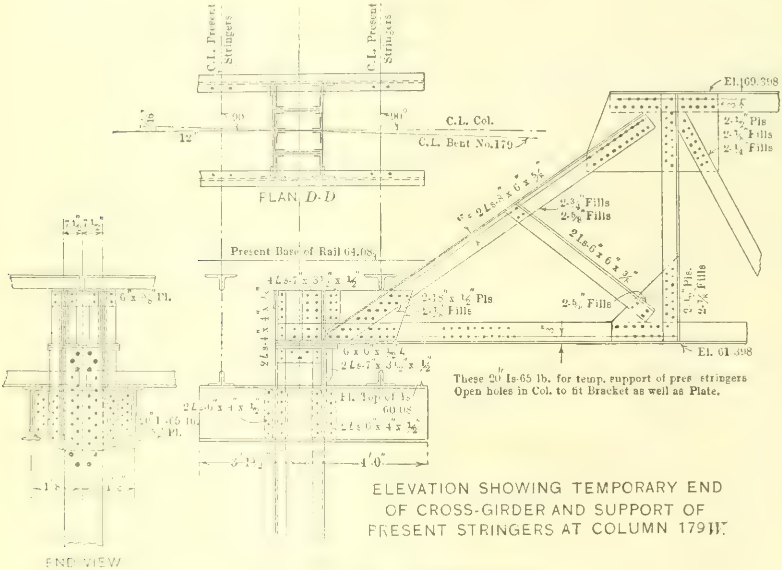
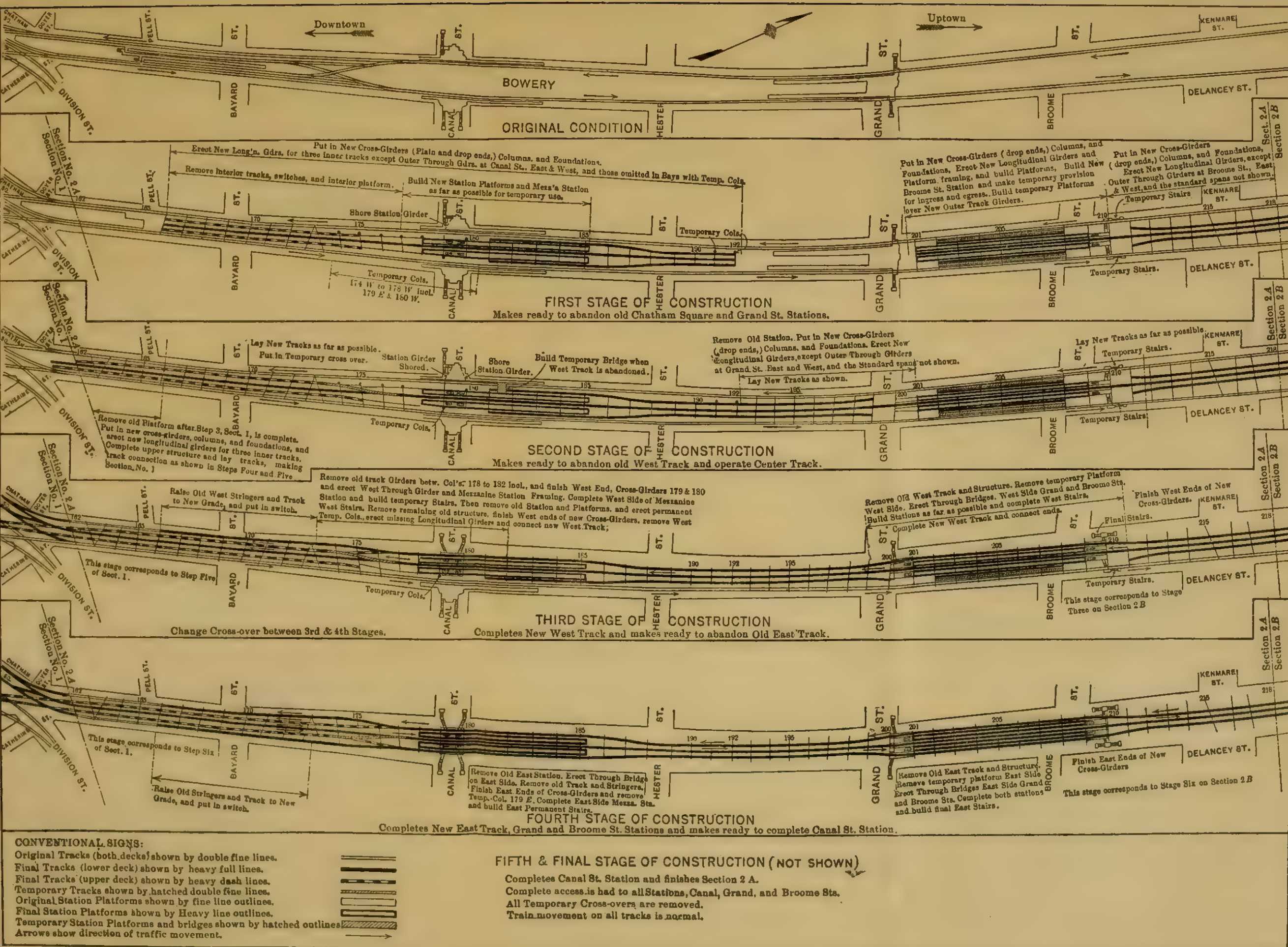


FIG. 100.

columns rested on a grillage of four I-beams, set directly on top of the street pavement, in the direction of the traffic, and completely encased in concrete. When the two bents, 191 and 192, were erected, including the track stringers, the derrick which had been used to place this work, was re-erected on top of the track stringers, and, when this work was completed, it was used to erect all the structure south of this point. At and south of Canal Street there were several bents which were to be supported permanently on sidewalk columns. As the new structure was several feet above the present one, parts only of the cross-girders were erected and temporary columns in the

PLATE XLIX.
PAPERS, AM. SOC. C. E.
DECEMBER, 1917.
GARDINER AND JOHANNESSON ON
ELEVATED RAILWAY IMPROVEMENTS.



roadway were provided to support these cross-girders. At one point —the south side of Canal Street—the street-railway tracks which curved around from the Bowery to Canal Street, did not permit the placing of a temporary column in the street. The cross-girder, there-

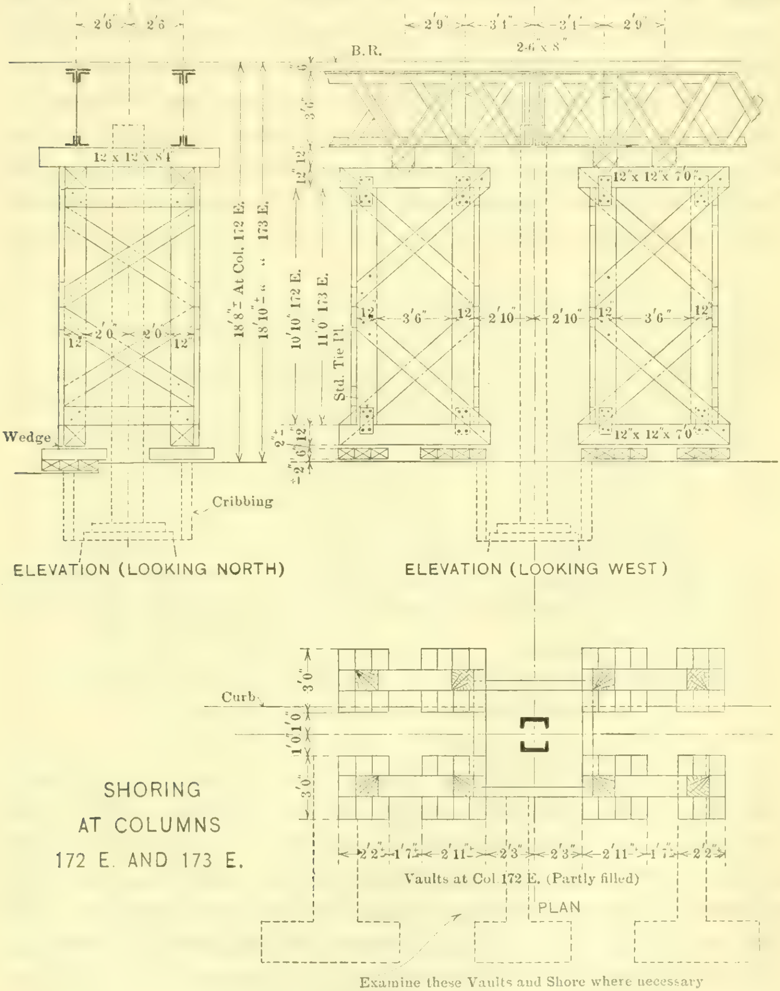
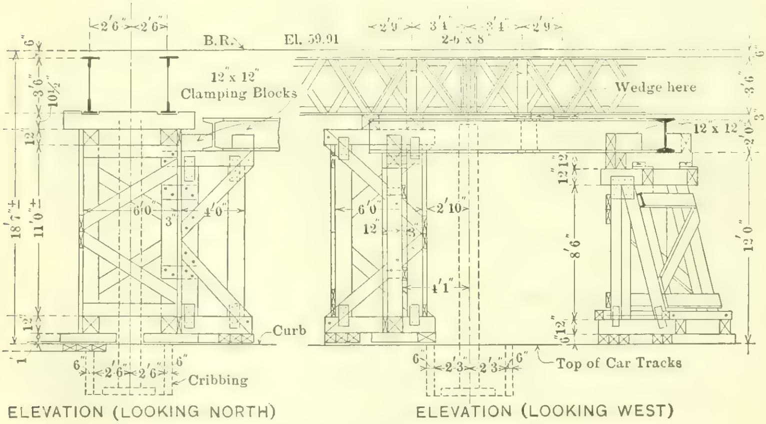


Fig. 101.

fore, was provided with a temporary shallow end which was supported on the permanent column on the sidewalk, as shown by Fig. 100. To support the existing track stringers, a temporary bracket was riveted to the new column shaft, as shown. From Bent 173 south, the new

columns were all on the sidewalk, and replaced the existing columns. The new lower-track level approached the level of the existing tracks, and the cross-girders, therefore, were designed to be placed below and without interfering with those tracks. The track stringers were shored while this work was being done, as shown by Fig. 101. First, an excavation was made to an elevation slightly below the bottom of the base of the new column. This excavation was lined solidly with 6 by 8-in. timbers to make it safe to support the shoring directly on the edge, which was desired in order to support the track stringers as near as possible to the connection to the column. The inside of the lined excavation was approximately 4½ by 5 ft., which permitted sufficient working space to remove the old column, to cut down the existing



SHORING AT COLUMN NO. 171 EAST

FIG. 102.

foundation to match the new columns, which were generally set deeper than the existing ones, and to set the new columns. The shoring consisted of two four-post framed bents set on short blocks directly on top of the street and sidewalk pavement. Blocking was placed between the top of the shoring bents and the track stringers, which were reinforced with vertical wooden web members, in order to transmit the web stresses directly to the shoring. The load was transferred from the existing column to the shoring by driving wedges between the bottom of the shoring bents and the blocking in the street. Special shoring had to be provided at certain places where street-car tracks interfered with the usual method. Fig. 102 shows the method of shoring at Bent 171, where a street-railway

track curves into the Bowery. On one side of the column a standard shoring bent was used; on the other side two shoring bents were placed so as to clear the car track, and the structure was carried by two 24-in. I-beams, resting on the shoring bents. The clearance below these I-beams was sufficient for the street cars to pass, but not sufficient for ordinary street traffic.

Another derrick was started north of the Grand Street Station. This derrick was erected on a platform, furnished by first erecting Bents 201 and 202, including the stringers between these bents. This derrick moved north, erecting the steel until stopped by an injunction, some property owners objecting to having the columns placed on the sidewalk, although the certificate under which the work was done did not permit their being placed in the street. The injunction was eventually removed, but, in the meantime, the derrick was taken down and moved to the north end of Section No. 2-B, at Fourth Street, and the steel erection was completed by this derrick moving south to the point where the erection had been stopped.

From Bent 201 to the north end of the section, the columns of the new bents were all placed on the sidewalks. The columns of the old structure, which were generally not connected with cross-girders, were originally placed without attempting to square up their location across the street. If the old column locations had been retained in the new structure, all the cross-girders would have been skewed considerably in relation to the track stringers. The old column locations therefore, were retained only on one side of the street, and, on the other side, the new columns were placed directly opposite, without reference to the existing locations. The new cross-girders were deep in the center portion, but the ends were made shallow enough to be erected without the top flange interfering with the existing operating tracks. Where the old columns had to be removed, in order to place the new columns and cross-girders, the track structures were shored in a manner similar to that described for the bents from Bent 173 south. On the other side of the street, the new cross-girders intersected the existing track stringers, which, therefore, had to be cut prior to the erection of the cross-girders. It will be remembered that the new foundations for the columns supporting these new cross-girders, had been constructed in advance of this work. These foundations were uncovered sufficiently to set the new columns, and the pit

was lined with 6 by 8-in. timbers, as described. The new column was then set in place, and a four-post framed shoring bent was built around it, as shown by Fig. 103, supported on blocking laid on the street surface. The shoring bent was made to carry the load of the stringers by driving wedges between the top of the blocking and the bottom of the shoring bent. The work to be done consisted of cutting the near-side track stringers to make room for the new cross-girders. The far-side track stringer was not affected, as the cross-girder did not extend far enough to interfere with it. After the shoring bent was

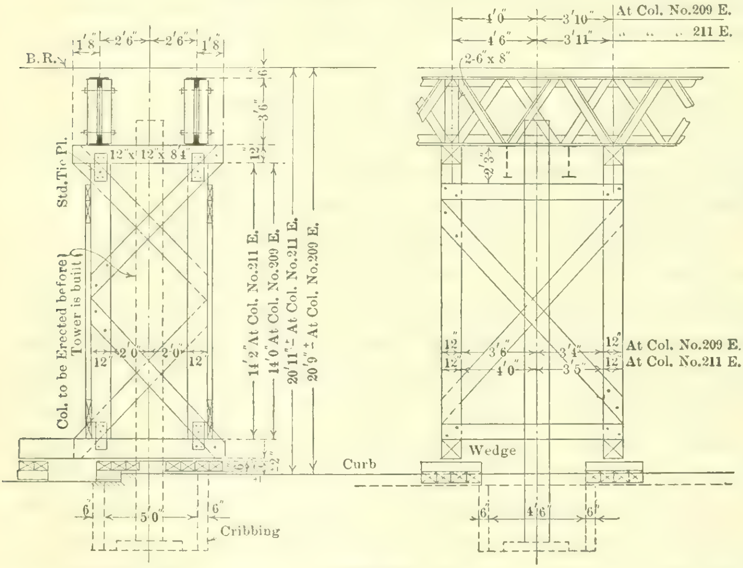


FIG. 103.

in place and wedged up, the track stringer was cut and the new cross-girder was set in place on top of the new column. The stringer ends were then supported on a temporary seat on the new column, and the shoring was released.

Before the new structure could be put in service, the portion of the structure occupied by the existing Grand Street Station had to be placed. This could not be done until the new Grand Street Station had been sufficiently completed and provided with temporary platforms to serve the existing tracks. This was accomplished by the traveler placing the new outside track stringers (which could not be

put in their permanent position, as they would then interfere with the running tracks), in a temporary position adjacent to the running tracks. As the new track level was purposely placed at the correct elevation above the existing track level, the new track stringers in their temporary positions were just suitable for supporting the temporary platforms. The new mezzanine station building of the Grand Street Station at the Broome Street end was sufficiently completed to be put in service, and was connected to the temporary platforms. The old Grand Street Station was then abandoned and demolished, and the new steel structure was completed throughout the section sufficiently to lay the center track. The new west island platforms of the station at Grand Street and Canal Street were then completed. At the Canal Street Station it was originally intended to gain access to the new platforms by a temporary bridge from the old platform after the south-bound track had been abandoned. It was found, however, that it would be possible to place a temporary stairway at the east side of the roadway close to the curb at the north side of Canal Street. This stairway, therefore, was placed connecting to the new mezzanine station at Canal Street, which was sufficiently completed to be placed in service. The scheme was to use temporarily the new center track for south-bound service, so that the existing south-bound track could be abandoned and the new structure on the west side completed. The new permanent connection, therefore, was made between the new center track and the local south-bound track to City Hall at Chatham Square just south of Section No. 2-A, and a temporary cross-over was provided between the center track and the new south-bound local track to South Ferry, just south of the Canal Street Station. This local track to South Ferry rises above the grade of the existing tracks and continues on the upper-grade structure, which had been constructed as part of Section No. 1 through the New Bowery and had been completed ready for operation at this time. At the same time the new center track had been completed through Section No. 2-B to Fourth Street, and the local traffic was temporarily carried on the newly constructed tracks from Fourth Street to Chatham Square. This left the existing structures on the west side of the Bowery free to be removed, which was done, and the new structure was completed on this side of the street. This work included the erection of the sidewalk columns and the com-

pletion of the cross-girders, the erection of which had been interfered with by the existing structure, and also the removal of the temporary columns on the west side of the roadway. When this work had been completed and the new south-bound track had been laid, the temporary cross-over south of Canal Street was removed, and the permanent connection was made to the new south-bound track to South Ferry. The south-bound traffic was then diverted from the center track to the new permanent south-bound track and the north-bound traffic between Chatham Square and Fourth Street to the center track, which involved the completion of the permanent track connections at Chatham Square and the placing of a temporary cross-over south of Canal Street to connect the new upper-deck track from South Ferry with the center track. The work remaining to be done to complete the structure on the east side of the Bowery was then completed, the temporary cross-over south of Canal Street was removed, and the local north-bound service was transferred to its permanent track.

Steel Erection.—Section No. 2-B.—In Section No. 2-B, which extended, on the Bowery, from the north end of Section No. 2-A, near Delancey Street, to Fifth Street, the new structure was placed in the middle of the roadway; the existing structure was on the sidewalk. There was, therefore, practically no interference with the existing tracks during the steel erection, except near Fifth Street, where a connection was made to the existing structure, which, at this point, is in the roadway. Following a similar procedure to that described under Section No. 2-A, two bents and their track stringers were erected near Fourth Street with a derrick set up in the street. The derrick was then erected on this platform and moved south, erecting the steel structure as it proceeded.

Later, another derrick was set up on the same platform to erect the steel north of this point. The new track level was several inches higher than the existing level, and, as it became necessary to make temporary connections with the running tracks, these were raised to make the connections on a proper grade. The raising was done by placing four-post shoring bents under the track stringers, and jacking and wedging the track stringers to the proper level after the column had been disconnected.

Fig. 98 shows the erection in progress in the vicinity of the Houston Street Station.

Steel Erection.—Section No. 3.—The first work of steel erection on this section was the reinforcement of the column tops, as described under the heading, "Steel Design." The reinforcement of existing lattice cross-girders between 33d and 35th Streets was commenced soon thereafter. It consisted of replacing the existing lattice members with web-plates. While this work was being done, the track stringers were carried on shorings, as shown by Fig. 104. On each side of the columns supporting a cross-girder was placed a four-post shoring bent with the posts tapering. The shoring bents rested on blocking laid on the surface of the street; this blocking was leveled up with wedges, and a lean, liquid mixture of sand and cement was poured to fill the space between it and the street surface. On top of each pair of shoring bents was placed two 24-in. I-beams crosswise under the structure. The track stringers were supported by blocking on these I-beams, and the load was transferred to the shoring by wedging under the track stringers. The reinforcing web-plates, which were generally about 14 ft. long, were raised to their proper level between the building line and the structure, and were then gradually worked in between the ends of the track stringers. At the 34th Street Station, where the station structure interfered with the placing of the plates in this manner, they were erected in shorter pieces between the tracks.

After this work was completed, the cross-girders between 12th and 16th Streets were reinforced. At this location the existing cross-girders were made with two end pieces and a center piece. The existing end pieces were removed and replaced with new ones, and the center piece was reinforced with a new web-plate. In addition to this the center-track stringers were remodeled to change the existing connections to the cross-girders to seat the connections. While this work was going on, a traveler was erected just north of 35th Street to place the new track stringers and cross-girders north of this point. In order to place the new cross-girders the existing track stringers were disconnected and carried on shoring until the erection of the new steelwork was completed. Two bents were shored at the same time, and the shoring was moved forward as soon as the work permitted its release. The work of erecting the steelwork for the hump stations at 9th, 23d, 42d, and 106th Streets was done by travelers starting at one end of the

approach and moving forward, erecting all steelwork, except some for the express platform, which generally was erected with a jinnywink operating behind the traveler. Fig. 125 shows the erection of the steelwork for the express station and track at the 9th Street Station as well as the derrick and the jinnywink in operation.

Steel Erection.—Section No. 4-A.—The steel erection for this section, which comprised the Second Avenue Line from Chatham Square to 116th Street, involved mainly the placing of new track stringers for the center track, which work was done with traveling derricks moving forward on the stringers set by the traveler. At the 86th Street Station, it was originally intended to have no express station, but after the center track stringers had been designed and fabricated to be placed on the level of the existing tracks, the Public Service Commission ordered an express station to be built at this point. This station is of the hump type. Another hump station was constructed at 14th Street, and a mezzanine station was built at 42d Street. The existing structure at this point was not sufficiently high to provide head-room under it for a mezzanine station. The track structure, therefore, was first raised to the necessary height, which amounted to a maximum of 15 in. The raising of the structure was accomplished by placing blocks between the top of the cross-girders and the under side of the lips of the track stringers. The reconstruction of the station was commenced on the west side of the structure, and necessitated the abandonment of the existing south-bound track throughout the portion to be constructed. The south-bound traffic was turned into the existing center track north and south of the station, and, in order to maintain access to the station, a temporary platform was constructed over the abandoned track adjacent to the existing platform and extending north for 260 ft. from the center of 42d Street. In order not to interfere with traffic, the timber bents supporting this platform were cut and framed in advance, and the platform was placed one night between 1 and 5 A. M., during which time no trains were operated on the Second Avenue Line.

The south end of the old south-bound platform was then removed, the existing track girders of the south-bound track were shifted into their new positions, the south half of the new west island platform was constructed, and, in order to make this platform long enough to serve the traffic, a temporary timber extension was built at the

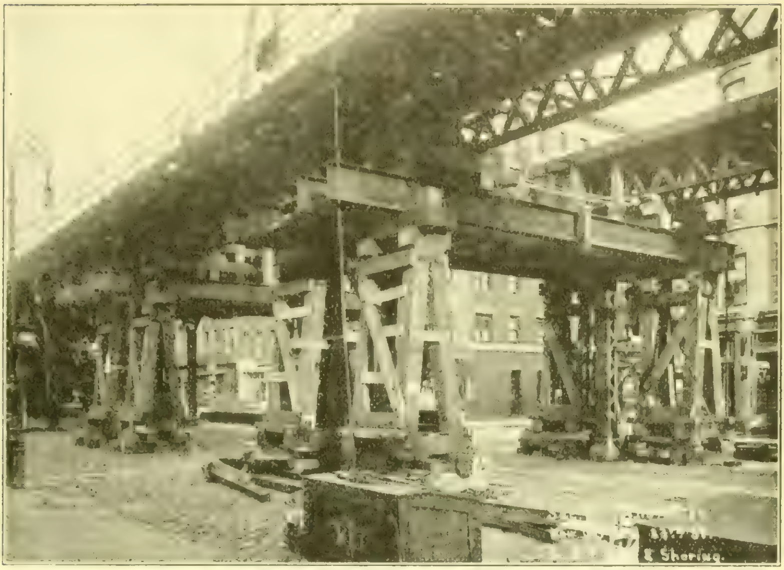


FIG. 104.—SHORING IN THIRD AVENUE AT 33D STREET.

FIG. 105.—STATION ON THIRD AVENUE AT 42D STREET DURING RECONSTRUCTION.

south end. Access to the platform was obtained by a stairway to the new mezzanine station, which was partly completed and connected to the street by a stairway. When this work was completed and ready for operation, the existing south-bound station was abandoned for traffic, as well as the temporary platform north of 42d Street, and the south-bound trains, still running on the center track, were stopped at the new island platform. The platform construction and the west station building were then removed, and the relocation of the south-bound track and the construction of the west island platform were completed. South-bound traffic was then run on the new permanent south-bound track, and the north-bound traffic was diverted to the center track, the new west island platform serving both tracks. This left the east side of the structure clear of traffic and permitted the relocation of the north-bound track and the completion of the construction of the east island platform. Fig. 105 shows the temporary platform north of 42d Street in place and the center track being used for south-bound service; half of the station building has been removed. The new mezzanine station is placed in the span, just south of the center line of 42d Street, and is supported by the through bridge girders shown. These girders also support the track structure and the platforms. Fig. 106 shows the southerly half of the new west island platform in course of construction.

At the 92d Street Station the existing platform was centered between the two running tracks, and was removed to make room for the new express track. To replace this platform, two side platforms were constructed, and, as the structure at this point is very high, a mezzanine station was built under the track structure. The existing cross-girders were first lengthened, and the new platforms and mezzanine were constructed. When these platforms and the stairs connecting them with the mezzanine station were completed, the old center platform was removed.

Steel Erection.—Section No. 5-A.—In this section, which comprised the Third Avenue Line between 116th Street and the Harlem River, the approach to the upper-deck express station commenced near 121st Street, and, as the express track was to connect with the upper deck of the Harlem River Bridge, it remained elevated throughout the section. Before the upper-deck structure was erected, the existing cross-girders and columns supporting the lower-deck

structure in Third Avenue between 123d and 128th Streets were replaced. In order to support the track stringers while the cross-girders and columns were being removed, two 24-in. I-beams were placed under the structure on each side of the cross-girder and supported on timber shoring bents, in a manner similar to that described under Section No. 3. As it was the intention to scrap the old material, no attempt was made to preserve the members intact, and the removal of the old structure, therefore, was facilitated by burning it, with a blaugas flame, into pieces convenient to handle. After the old columns and cross-girder of a bent had been removed, the new cross-girder (which was shipped loose, with the top flange and such web-angles as might interfere) was hoisted into a position directly below that which it was to occupy permanently, and was supported in this position temporarily by the shoring. It was then worked into place by jacking and blocking through the narrow space between the ends of the track stringers. The top flange-angles were afterward pushed in from the side of the structure between the track stringers and the ties, and the work was completed by riveting the pieces together. Whenever the stringer ends were to be remodeled, this was done in conjunction with the erection of the new cross-girders. The columns, which had detachable bases, were set after the cross-girder was in place, the procedure being first to set the loose base over the anchor-bolts and then slip the column shaft in between the base and the cross-girder. As the structure was generally jacked up a little above the final elevation, in order to carry the load of the structure on the shoring, little difficulty was experienced in putting the columns in place.

South of the 125th Street Station, the existing cross-girders and columns were retained, but the cross-girders were reinforced and the ends of the center-track stringers were remodeled. As this work was done while traffic was running over the center track, the structure had to be shored. The method of shoring is shown on Fig. 108. The shoring bents had four posts, all in one plane, and were secured to the columns by yokes. These bents were not designed originally for this special purpose, but had been used on another section. The shoring girders which, as usual, consisted of two pairs of 24-in. I-beams, rested on top of these bents and supported the track stringers.

FIG. 106.—STATION IN SECOND AVENUE AT 42D STREET, STRUCTURE ABOVE MEZZANINE.

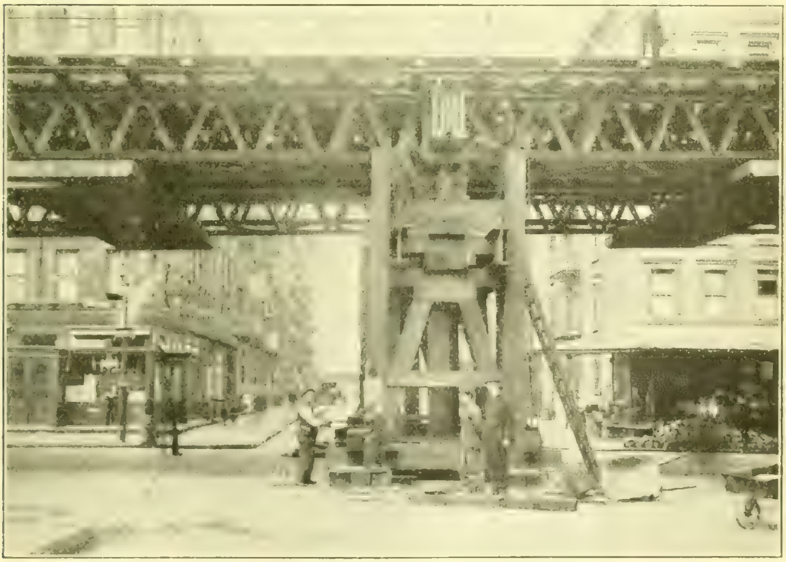
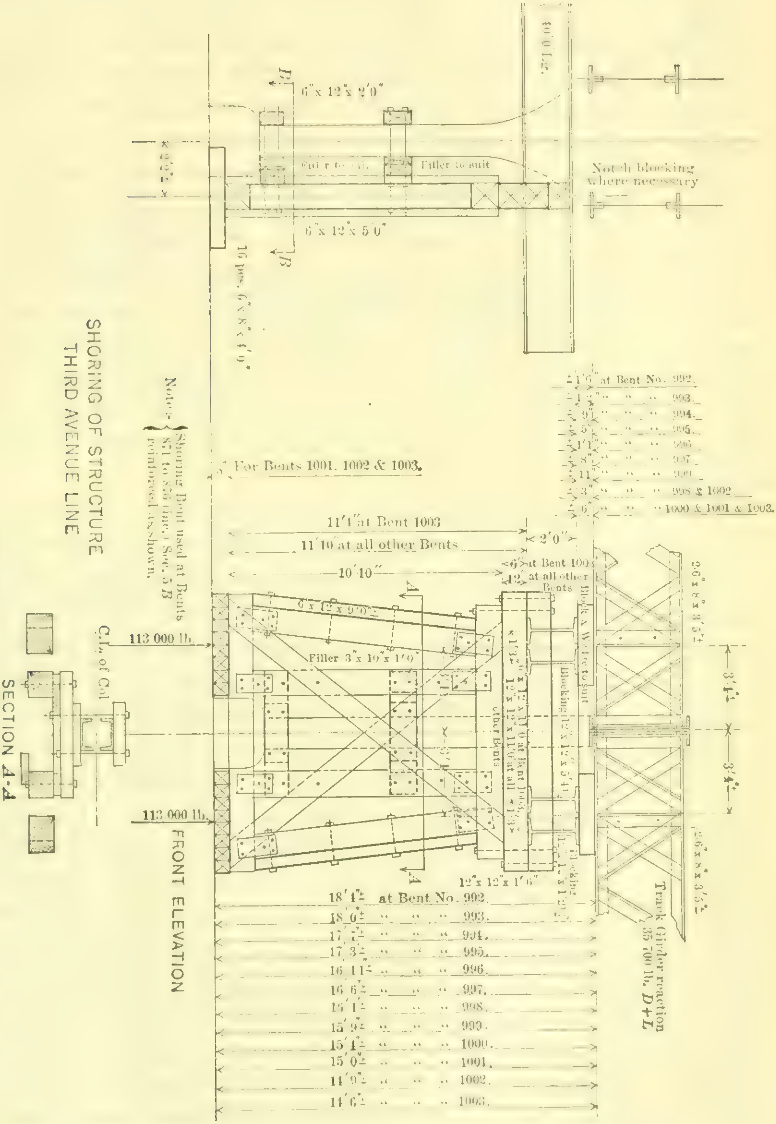


FIG. 107.—RAISING STRUCTURE IN SECOND AVENUE WITH HYDRAULIC JACKS.



SHORING OF STRUCTURE
THIRD AVENUE LINE

FIG. 108.

The derrick which erected the upper-deck structure, was set up at the south end of the upper-deck approach, and erected the steel structure complete as it proceeded, until it reached 128th Street. At this point the length of the new upper-deck span is about 90 ft., and the girders weighed about 17 tons each. The derrick was not able to handle these girders. Another and heavier derrick was erected, therefore, on top of a tower construction built from the street level to the level of the lower deck. This derrick raised each girder from the street and placed it in position on the new cross-girders, which were also raised by this derrick. As it was the invariable rule not to permit a train to pass under steel in the process of erection, while suspended from a derrick, or while not securely fastened to its supports, the erection of these girders delayed the traffic, the delay in each case being about 15 min. After this bridge structure had been erected, the auxiliary derrick was dismantled, and the first derrick proceeded to erect the work through to the Harlem River Bridge.

Steel Erection.—Section No. 5-B.—This section extended from 116th to 129th Street on Second Avenue. To provide the proper grade for the connection to the two decks of the new Harlem River Bridge, the complete structure had to be raised for a length of 1 387 ft., the maximum rise being 7.5 ft. As it was essential that traffic should be maintained while the structure was being raised, and that the grades should at all times be reasonable, a complete plan was worked out in advance of the work, giving the location of the points to be raised at one time, the rise at each point, and the sequence of the lifting operations. In the preliminary studies it was, as seemed reasonable, first attempted to start the lifting operations at the point of the line where the final rise was greatest, that is, at the middle of the portion of the structure to be raised, but it was soon found that no satisfactory grade could be maintained starting at this point. The scheme decided on is shown on the upper part of Plate L. The lifting operations were started at the south end of the new grade, and usually five bents were raised during one operation. The maximum rise during any one operation was limited to 6 in. The second lift was partly superimposed on the first one, but extended over three additional bents, and the work was continued in this manner until five operations were completed, and the structure for about half the length had been raised

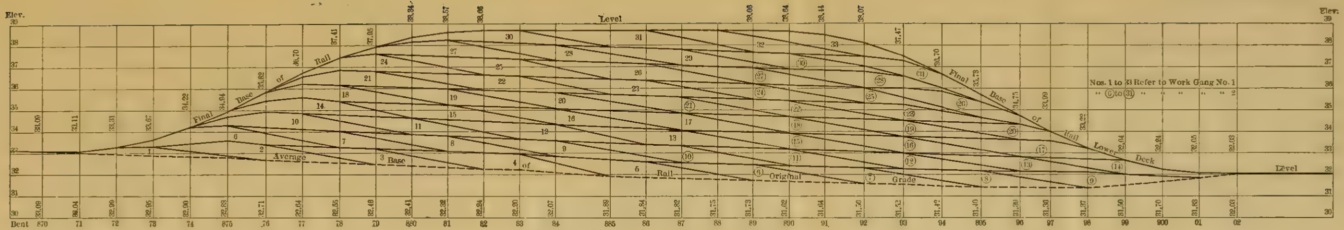
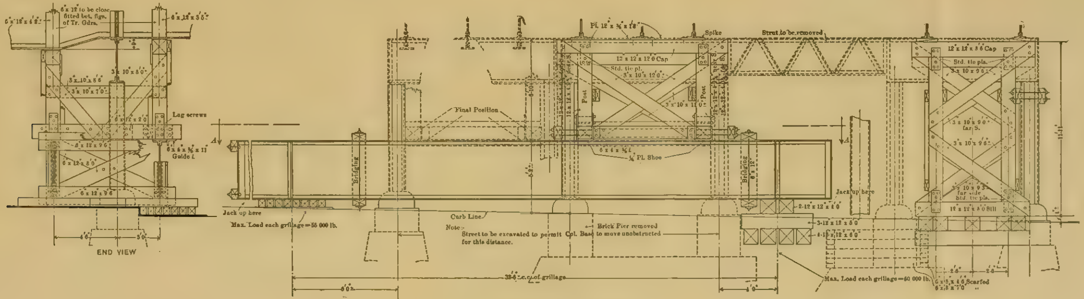


DIAGRAM OF SUCCESSIVE DAILY JACKING OPERATIONS



SHORING OF STRUCTURE, THIRD AVENUE LINE

a maximum of about 6 in. Another set of jacks was then started operating, one set continuing north from the location of the fifth operation, and the other set starting at the south end of the grade. The effect of this method of raising the structure was actually to raise it in steps the full length of the rise, each step being approximately 6 in. high. The numbers on the diagram indicate the sequence of the operations, and the numbers enclosed in a circle indicate the operations of the second set of jacks. As this second set was not working during the five first operations of the first set, the first operation of this set is numbered 6, to correspond to the sixth operation of the first set of jacks. It was intended originally to raise the structure between 1 and 5 A. M., when no trains are running on this line, and it was intended to complete one operation only during each night, but, as work of this kind would be very unsatisfactory to perform by artificial light, it was decided to do it during the day, and the jacking operations were completed in 17 working days, without interfering with the operation of trains.

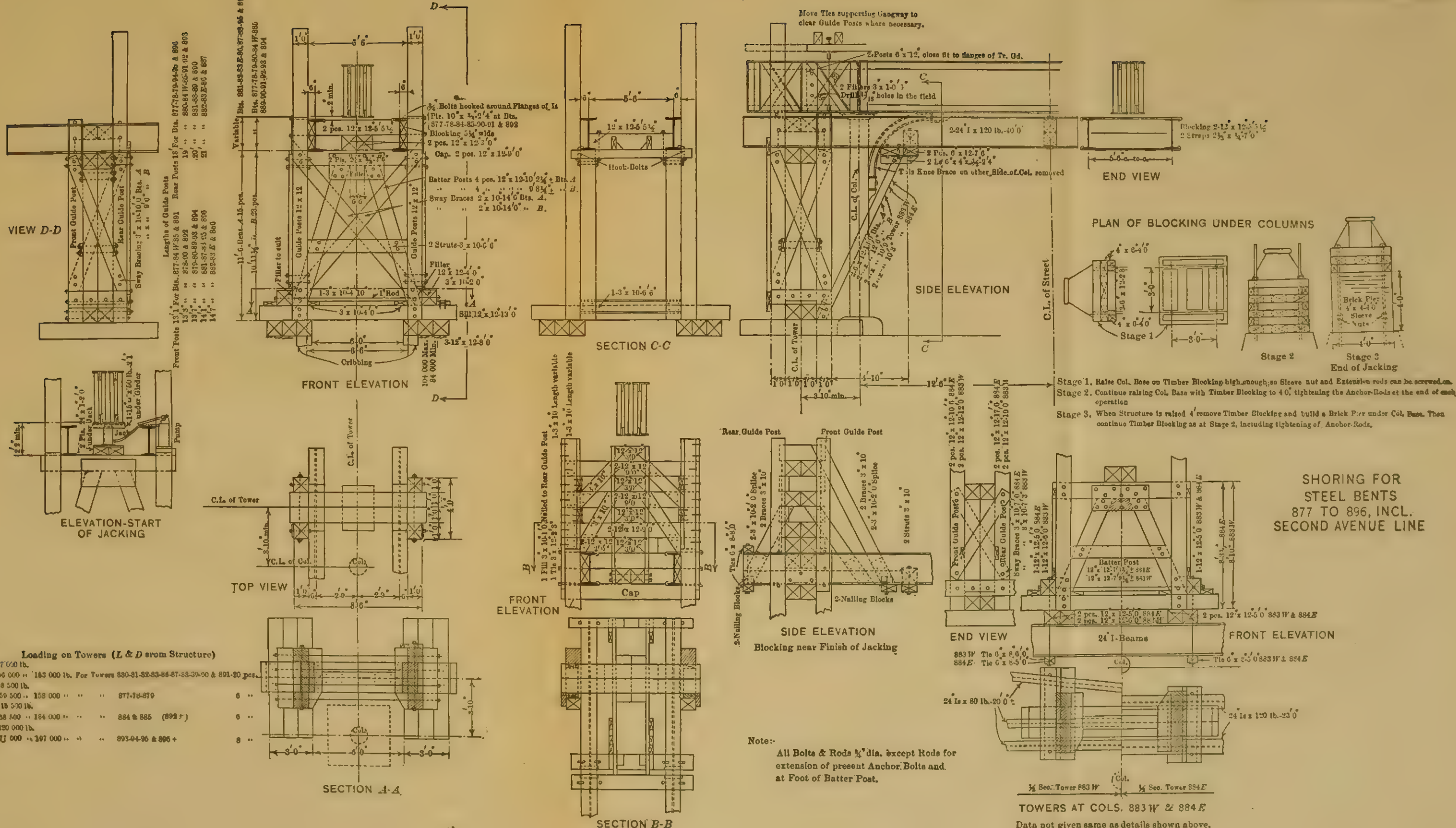
At both ends of the structure raised, where the lift was less than 3 ft. 6 in., that is, where the track level could be raised to its proper grade by raising the track stringers only, without bringing the bottom of the stringers above the top of the cross-girders, the track stringers only were raised. For the remainder of the length, which altogether included twenty bents, the complete structure, including cross-girders and columns, was raised. Where the track stringers only were raised, they were supported during the work by shoring. A four-post bent with all the posts in the same plane and with the two outside posts battered, was set on a grillage of 12 by 12-in. timbers laid on the street surface and grouted. The bent was placed outside of the columns, in order not to interfere with the street-car service, the tracks of which were between the columns. The bent was tied to the column by two yokes, as shown. On top of two such bents, attached at the columns supporting the same cross-girder, were placed four 24-in. I-beams, two on each side of the cross-girder. These I-beams had been used previously for shoring girders during the foundation work on other sections, and, as it was desired not to punch holes in them, in order not to destroy their value for any future use, the I-beams were held in place by wood blocks and steel straps. The jacks used for raising the girders were set on top of these I-beams, and,

during the jacking operation, the space between the lip of the stringers and the top of the cross-girder was continuously kept filled with blocking and wedges, in order to prevent any dropping of the stringers due to failure of the jacks. When a jacking operation was completed, the space between the top of the I-beams and the bottom of the track stringers was also filled with blocking, so as to have double security against dropping of the stringers.

Where the structure was to be raised complete, the method of procedure was as follows: First, a pit was dug around the base of each column, deep enough to expose the base. These pits were 6 ft. square, after being lined solidly with 12-in. timbers. A double A-frame, of 12 by 12-in. timbers, as shown on Plate LI, was then placed under the ends of the cross-girders, outside the columns, on timber blocking laid on the street surface, and grouted. A working platform was provided on top of the A-frames and 12 by 12-in. girder posts at each corner of the platforms. These girder posts were to keep the blocking in place, and will be described later, but the inside ones could not be carried to their full height before the structure was raised, as they would interfere with the track stringers in their original position. They were arranged, therefore, so that they could be spliced afterward. To stiffen the bents laterally during the work, two 24-in. I-beams connected the two shoring bents under the same cross-girder, and were braced by brackets to them.

When all the structure to be raised was shored, the nuts of the anchor-bolts in the bents to be raised during an operation were loosened, and the jacks, which were set on top of the working platforms, were started. The rise of the structure was followed up with wedges and blocking, both under the cross-girder and under the bases of the columns. When the structure was raised high enough to permit it, sleeve nuts were screwed on to the anchor-bolts, and extension anchor-bolts, consisting of 2-in. rods, threaded their full length, were added. Afterward, the structure was secured to the foundations after each jacking operation by tightening the nuts on the extension bolts. On the top of the shoring bents, the working platform was gradually raised by a blocking of 12 by 12-in. timbers secured to the shoring by the guide-posts and by braces to the I-beams, as shown.

Two of the shoring bents had to be placed immediately above public service manhole chambers in the street. They were supported, there-


VIEW D-D
FRONT ELEVATION
SECTION C-C
SIDE ELEVATION
END VIEW
PLAN OF BLOCKING UNDER COLUMNS
Stage 1
Stage 2
Stage 3
End of Jacking
Stage 1. Raise Col. Base on Timber Blocking high enough so Sleeve nut and Extension rods can be screwed on.
Stage 2. Continue raising Col. Base with Timber Blocking to 4 0, tightening the Anchor-Rods at the end of each operation
Stage 3. When Structure is raised 4' remove Timber Blocking and build a Brick Pier under Col. Base. Then continue Timber Blocking as at Stage 2, including tightening of Anchor-Rods.
ELEVATION-START OF JACKING
TOP VIEW
SECTION A-A
FRONT ELEVATION
SECTION B-B
SIDE ELEVATION
Blocking near Finish of Jacking
END VIEW
FRONT ELEVATION
SHORING FOR STEEL BENTS 877 TO 896, INCL. SECOND AVENUE LINE
Loading on Towers (L & D from Structure)
Note:- All Bolts & Rods ¾" dia. except Rods for extension of present Anchor Bolts and at Foot of Batter Post.
TOWERS AT COLS. 883 W & 884 E
Data not given same as details shown above.

fore, on shoring girders, which removed the load from the roof of the chambers and also permitted the necessary access to them.

Fig. 107 shows a shoring bent during a jacking operation. To raise the track stringers, only 25-ton ratchet jacks were used, one under the end of each pair of stringers. Where the entire structure was raised, 60 and 100-ton hydraulic jacks were used.

With the structure was raised a signal tower and the southerly half of the platforms of the existing station at 127th Street. The track stringers in front of the remainder of the platforms were raised, and, in order to retain them in service, narrow platforms were built on top of the existing ones with steps leading down to the old level.

After the structure was raised to its proper elevation, new columns were placed where the cross-girders had been raised. The old columns were removed, and the new ones were placed by tackle attached to the structure girders. At the same time, the ends of the track stringers which had been raised were remodeled to rest on the cross-girders at their new elevation.

The next operation was the erection of the new station at 125th Street and the construction of the new connections to the Harlem River Bridge. Plate LII shows the different stages of this work. First, the north-bound traffic was turned into the center track south of the new station and returned to the existing north-bound track at a point just north of the new platforms, so as to serve the platform at the existing station at 127th Street. The deck of the abandoned track was then stripped, and the track stringers were moved sidewise to their new location, except across 125th Street, where the existing girders were removed and replaced with new through bridge girders. Three of these girders were placed to carry the east platform and the new north-bound track. The girders were erected with two gin-poles, one at each end of the girder. The track was then laid, and connection was made so that traffic could run over the new north-bound track. The south-bound traffic was then run over the center track, and the structure changes on the west side of the new 125th Street Station were made. When completed, the south-bound traffic was turned from the center track to the new south-bound track. Work, which during this period had been in progress on the two new island platforms, continued, and the erection of the mezzanine station was started. When sufficiently completed, and connected to the street, the east platform was placed

in service for both north and south-bound traffic, the center track again being used for south-bound traffic past the platform, and the existing station at 127th Street was abandoned and removed.

The outside tracks north of the new 125th Street Station were then moved to their new permanent position, the north-bound under traffic. The moving of the track stringers was done in all cases by sliding them on the top of the cross-girders, the work being done with 25-ton jacks.

The upper-deck structure was erected with a traveler, which was set up at the bottom of the incline just north of the new platforms of the station at 125th Street. This traveler erected all the steel for the upper deck as far as the junction with the Harlem River Bridge. Fig. 109 shows this work in progress. In the first two spans, old center-track stringers were used, with the ends remodeled; the remainder of the track stringers are new plate girders.

Steel Erection.—Section No. 5-C.—This section comprised the new Harlem River Bridge. On account of the necessity of maintaining traffic, the erection of the new bridge had to be carried out in such a manner that traffic over the existing bridge would not be interrupted, and it was decided, therefore, to build a new bridge on a pile platform at a convenient point in the river, so that, when the erection was completed, it could be floated into position. The first question to be settled, therefore, was the site for erecting the bridge. The Company owns a yard with a dock front adjacent to and south of the bridge, on the Manhattan side of the Harlem River. This dock front would have been a convenient location for building the bridge, but would have interfered with the use of the yard, and was also objected to by the War Department, on account of encroachment on the river. Just south of this yard, which was bounded on the south side by 128th Street, there was another dock front, set back of the United States bulkhead line. This, as well as the yard adjacent to it, was leased by the Company, and after obtaining the approval of the War Department, the space between the United States bulkhead line and this dock front was determined on as the site for the erection of the bridge. Fig. 111 shows the property leased and the location of the pile platform.

The bridge was built on a timber platform supported on piles. The platform was arranged so that the two shore spans could be

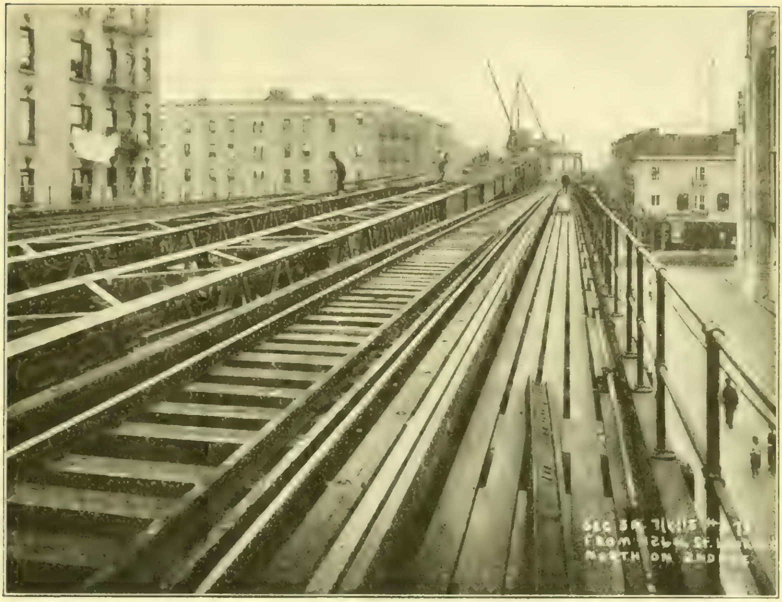
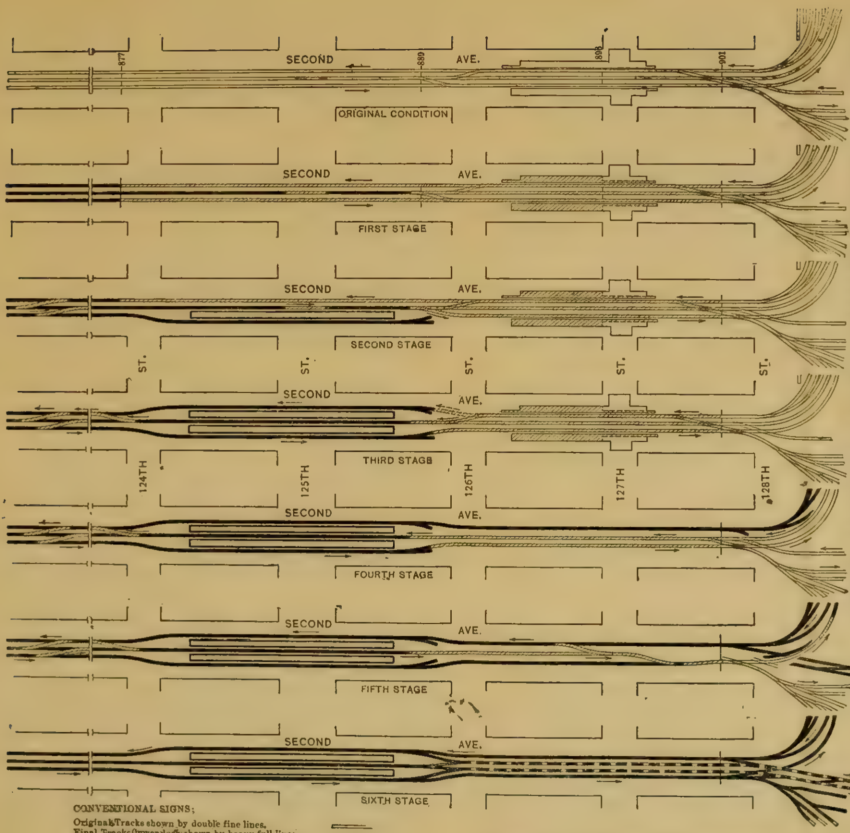


FIG. 109.—SECOND AVENUE LINE, NORTH OF 125TH STREET.

FIG. 110.—TEMPORARY PILE PLATFORM FOR ERECTION OF BRIDGE SPANS.



FIRST STAGE. Jack up Longitudinal Girders and Track to Permanent New Grade of Local Tracks from Bent 870 to Bent 901; also Cross-Girders from Bent 877 to 896. Jack up Platforms with Structure from Bent 896 South. Construct Narrow Temporary Platforms on Old Platforms to the New Grade of Tracks from Bent 896 North. Erect New Street Columns from Bent 877 to 896 and Sidewalk Columns for New Station at 125th St. and lengthen out Cross-Girders for Station. Make Alterations in Old Longitudinal Girders from Bent 870 to Bent 876.
Remove Platform from 128th St. to Harlem River Bridge. Construct New Foundations at Harlem River. Make all necessary Alterations in Old Structure to this point. This work may overlap next Three Stages.

SECOND STAGE. Put in Cross-Overs South of 123d Street, as shown. Turn North-Bound Traffic on to Center Track from 124th Street to 126th Street. Remove Old North-Bound Track over this Stretch. Move Old North-Bound Longitudinal Girders into new locations from Bent 878 to 888. Erect New Longitudinal Girders for East side from 877 to 890. Erect East Platform Girders and Build East Platform of 125th Street Station. Build East half of Station. Lay New North-Bound Track up to and including Switch, South of 126th Street.

THIRD STAGE. Put in Cross-Over South of 124th Street, as shown. Connect up Old and New North-Bound Track at 126th Street. Turn South-Bound Traffic on to Center Track form 126th Street to 124th Street. Turn North-Bound Traffic on to New East Track. Repeat operations of 2d Stage for West Track and West side of Structure.
Remove Switch at 126th Street. Connect up old South-Bound with New South-Bound at this point. Turn Traffic over this route. Remove Old Longitudinal Girders for Center Track at 125th Street Span. Erect New Floor-Beams and Stringers of Center Track and complete Station.

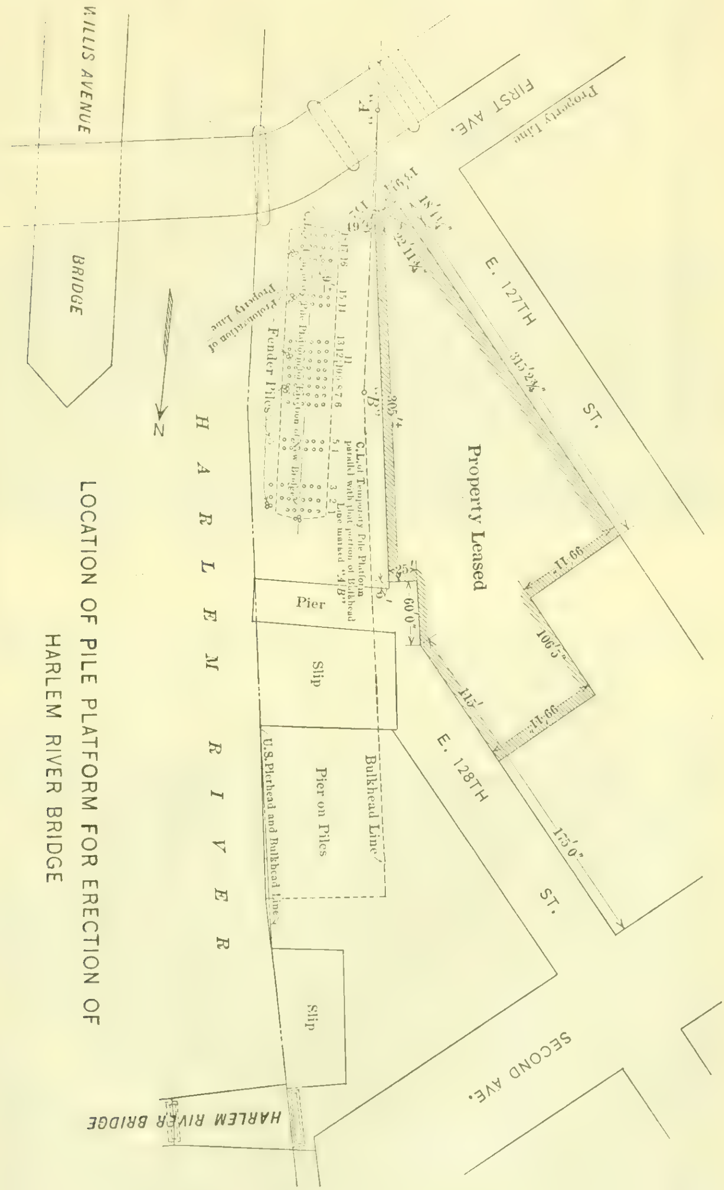
FOURTH STAGE. Move to New Station. Turn South-Bound Traffic on to Center Track past Old Station and past New Station. Tear out Old Station and Platforms. Erect New Longitudinal Girders for West side from Bent 890 to Bent 904. Move out Old Longitudinal Girders with track from Bent 891 to 902 and make new Connections with West Yard.

FIFTH STAGE. Turn South-Bound Traffic on to New South-Bound Track. Erect New Longitudinal Girders for East side from Bent 890 to Bent 902. Move out Old Longitudinal Girders with Track from Bent 891 to Bent 900. Turn North-Bound Traffic on to Center Track. Make temporary Cross-Over between East and West Tracks at 127th Street. Run all Bronx Traffic over Single East Track North of 128th Street. Remove Old Center Track from 127th Street around Curve. Lay New North and South-Bound Bronx Tracks for Lower Deck, as far as possible.

SIXTH STAGE. Complete New East Track between 127th Street and Harlem River Bridge. Turn North-Bound Traffic on to East Track. Remove Center Track from 126th Street to 127th Street. Connect South-Bound Track with South-Bound over Harlem River Bridge. Complete North-Bound Local Track from 127th Street North. Make Connection with and complete changes in East Yard. Erect and complete Upper Deck Structure.

CONVENTIONAL SIGNS;
Original Tracks shown by double fine lines.
Final Tracks (lower deck) shown by heavy full lines.
Final Tracks (upper deck) shown by heavy dash lines.
Temporary Tracks shown by hatched double fine lines.
Original Station Platforms shown by fine line outlines.
Final Station Platforms shown by heavy line outlines.
Temporary Station Platforms shown by hatched outlines.
Arrows show direction of traffic movement.

DIAGRAM OF PROGRESS OF ERECTION
SECOND AVENUE LINE



LOCATION OF PILE PLATFORM FOR ERECTION OF HARLEM RIVER BRIDGE

FIG. 111.

erected at the same time, and so that the swing span could be erected on the same platform after the shore spans had been removed.

Plate LIII shows the details of the platforms. The piles on which the platform was supported were driven to solid bottom and cut off and capped 4 ft. 6 in. above mean high water. The piles were braced with timbers between the top and low-water level, and, below this, additional cross-bracing was provided by wire cables looped around the pile at the one side of the bent and brought up diagonally and tied to the pile at the other side at low-water level.

The piles were arranged so that spaces were left between the bents wide enough to permit the passage of the barges which, when the erection was completed, were to float the bridge spans off the pile platform. On the top of the pile caps was erected a braced timber construction supporting 24-in. I-beams, which formed the deck, on which the bridge spans were erected. Plate LIII shows, in full lines, the position of the center span of the bridge on the platform, and, in dotted lines, the two end spans which were erected simultaneously. The south span was erected at the north (left) end of the platform. On account of the trusses of this span not being parallel, one end of the span was heavier than the other; this span, therefore, was not placed symmetrically on the platform, but in such a manner that the load of the span, when placed on the two barges which supported it when floated off the platform, would be distributed uniformly on the two barges.

The platform was arranged so that the bridge was erected at an elevation 2 ft. higher than that which it would occupy when supported on the bridge piers, in order to insure sufficient head-room when the bridge was floated in.

When the platform was completed, the erection of the steelwork for the north and south spans was commenced. This was done with a derrick-boat, the pile platform being about 40 ft. from the river bulkhead, so that the derrick-boat could erect the steel from either side of the platform. Fig. 110 shows the platform, with the two end spans nearly completed; the derrick-boat is also shown. Immediately after the steel erection was completed, the placing of the track structure was commenced. The connections of the existing tracks of the Second and Third Avenue Lines extended over the south shore spans, as shown by Fig. 112, which is a view of the old Harlem River Bridge.

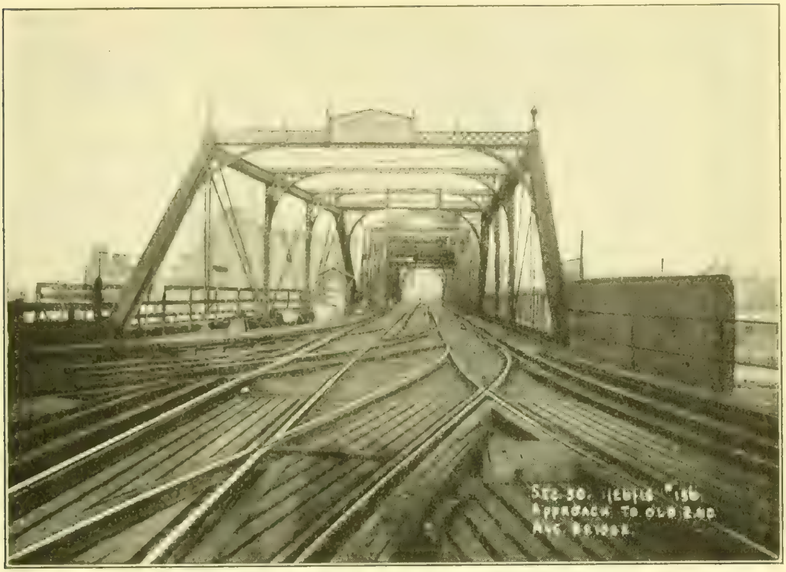


FIG. 112.—SOUTH APPROACH TO HARLEM RIVER BRIDGE.

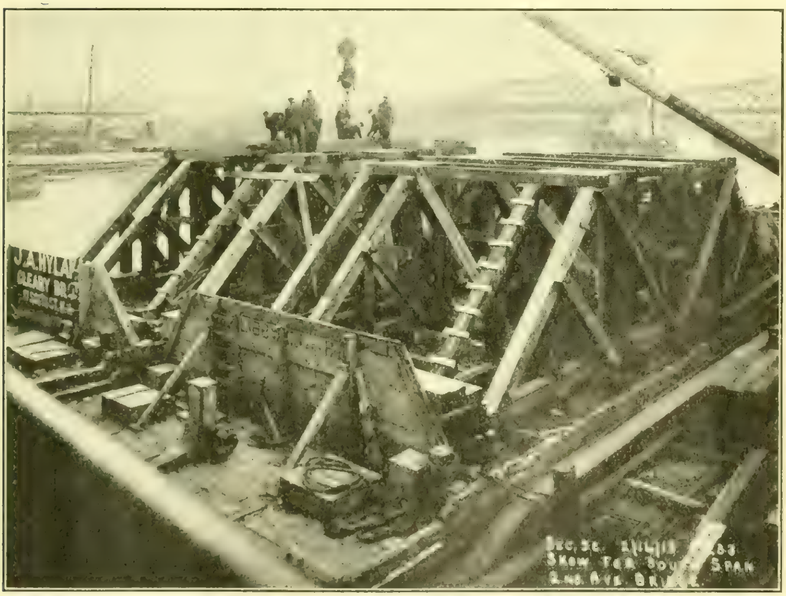
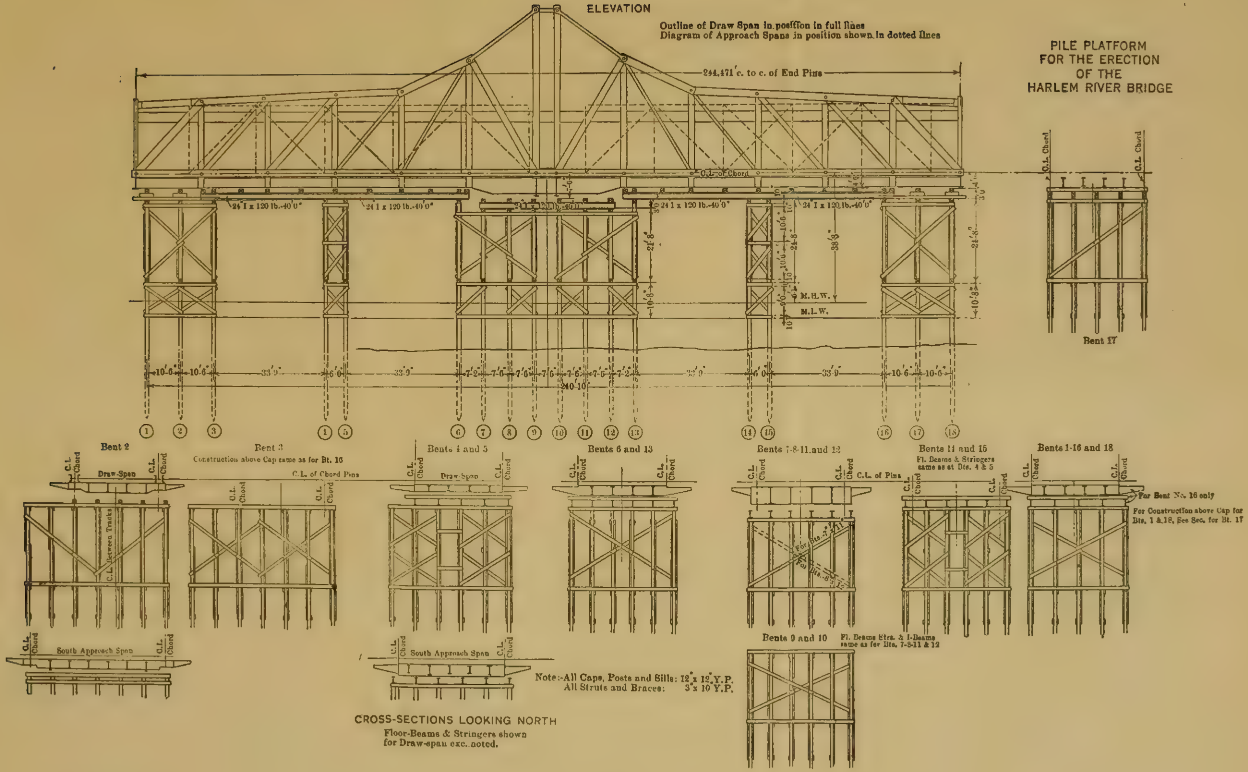


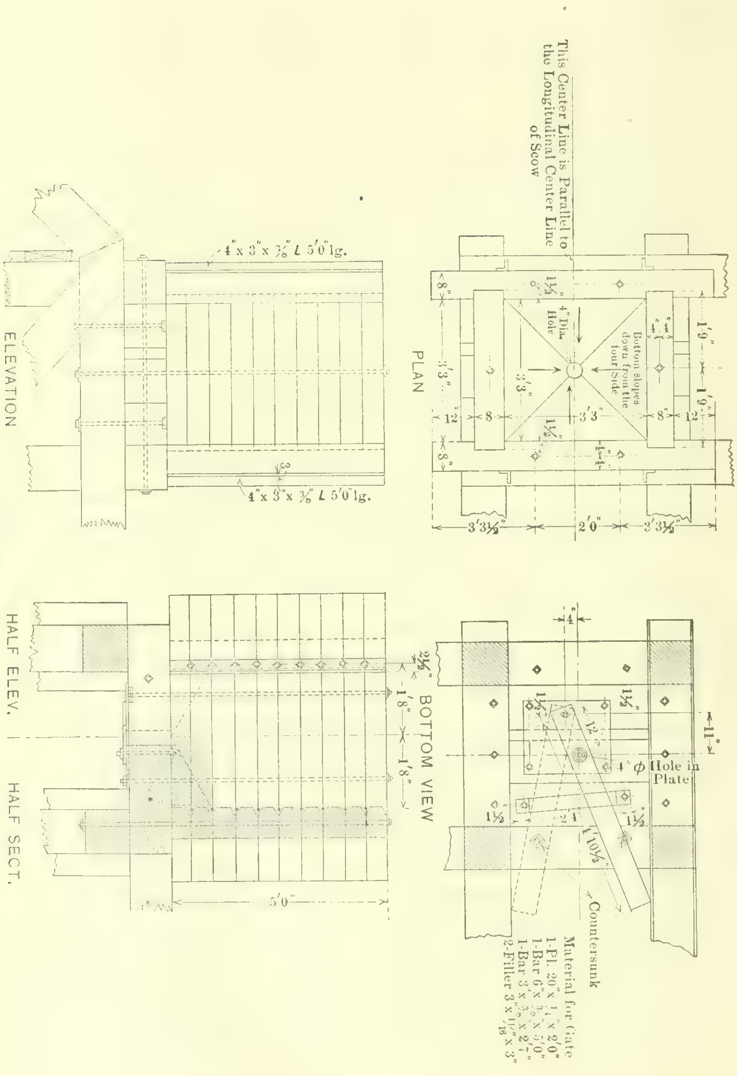
FIG. 113.—SCOWS WITH TIMBER TRUSSES FOR SUPPORTING NEW BRIDGE.


ELEVATION
Outline of Draw Span in position in full lines
Diagram of Approach Spans in position shown in dotted lines
244.471' c. to c. of End Pins
C.L. of Chord
24" I x 120 lb.-40'0"
M.H.W.
M.L.W.
240'10"
PILE PLATFORM
FOR THE ERECTION
OF THE
HARLEM RIVER BRIDGE
Bent 17
Bent 2
Draw-Span
C.L. Between Tracks
Bent 3
Construction above Cap same as for Bt. 16
C.L. of Chord Pins
Bents 4 and 5
Draw Span
Bents 6 and 13
Bents 7-8-11 and 12
C.L. of Pins
Bents 14 and 15
Fl. Beams & Stringers same as at Bts. 4 & 5
Bents 1-16 and 18
For Bent No. 16 only
For Construction above Cap for Bts. 1 & 18, See Sec. for Bt. 17
South Approach Span
Note:-All Caps, Posts and Sills: 12" x 12" Y.P.
All Struts and Braces: 3" x 10" Y.P.
Bents 9 and 10
Fl. Beams Strs. & I-Beams same as for Bts. 7-8-11 & 12
CROSS-SECTIONS LOOKING NORTH
Floor-Beams & Stringers shown for Draw-span exc. noted.

It was necessary that the alignment of the track laid on the new span should be an exact duplicate of that of the track existing on the old span, in order to avoid delays to the traffic after the old span had been replaced by the new. The existing track lay-out therefore, was surveyed carefully and reproduced on the new bridge span, and it may be stated that the new track matched exactly when the new bridge was placed.

At the same time, the scows, which were to float the new spans in place, were being made ready. These scows were 95 ft. long over all; their length at the water line, unloaded, was 80 ft.; their height above water, unloaded, was 6 ft., and their width was 29 ft. The scows were decked and well-braced, so that no additional bracing was necessary. On the deck of each scow was erected four timber trusses for the support of the bridge, as shown by Fig. 113.

The ranges of the tide in the Harlem River at the Harlem River Bridge are very irregular, and do not always correspond to those predicted. The tide at this point is affected by tides in both the North and East Rivers. The rising tide produces, what might be contrary to expectations, a southerly current, and the falling tide a northerly current in the river. The height of the tides, also, is affected considerably by the weather conditions. It was deemed advisable, therefore, to provide other means, in addition to the tide, to accomplish the lowering of the bridge spans to the pier, so that unforeseen tidal conditions should not prevent placing them at the desired time and thereby interrupt seriously the traffic across the bridge. The apparatus used was sand-jacks, the details of which are shown by Fig. 114. Each sand-jack consisted of a box, made of 6 by 8-in. timber, framed and tied together with bolts, and a plunger consisting of nine pieces of 12 by 12-in. timber. The box was originally 5 ft. high, but had a bottom sloping to the center, which reduced the effective height to 4 ft. The box was 3 ft. 3 in. square inside. In the center of the bottom of the box there was a hole, 4 in. in diameter that could be closed by a gate consisting of a 6 by $\frac{3}{4}$-in. steel bar, fastened at one end to the under side of the box in such a manner that by moving the other end of the bar sidewise, the opening could be closed or opened as desired. Six of these boxes were set on top of the timber trusses on each scow and bolted securely to the framework of the trusses. To the 24-in. I-beams, supporting the bridge spans on the



SAND BOX
FIG. 114.

timber platform, were bolted plungers, as shown by Fig. 115, in such positions that they would fit accurately over the boxes on the scows, when the latter were in their proper places, ready to lift the bridge spans off the platform. The plungers consisted of nine pieces of 12 by 12-in. timber, set on end, and bolted together so that their outside dimensions were 3 ft., leaving a clearance of $1\frac{1}{2}$ in. all around between the plungers and the boxes. The method of using the sand-jacks was to fill the boxes to the top with fine dry sand and let the plungers, which carried the bridge spans, be supported on top of the sand. When it was desired to lower the bridge, the gates at the bottom of the sand boxes were opened, and the sand flowed out, letting the plungers descend gradually in the boxes, and with them the bridge.

The south span of the bridge was first made ready to be moved. The time for the replacement was determined in conjunction with the Company's Traffic Department, which desired this to be the night between a Saturday and a Sunday, after the hour of 1 A. M., at which time the traffic over the bridge was lightest. As the rising tide was to be used for removing the old span, it was necessary to select a night when high tide occurred an hour or two after 1 P. M. After consulting the tide tables, Sunday morning, February 20th, 1915, was selected as the time for replacing the south span. As it was not deemed advisable to keep the span floating on the scows longer than necessary, on account of the possible danger of scows leaking and settling, or even sinking, it was decided to float the bridge span off the platform on the tide immediately preceding the one on which the span was to be set in place. This made the time for floating off the span, Saturday, February 19th, in the afternoon.

At 8 A. M. on this date, the two scows which were to carry the span were brought into their proper place, as shown by Fig. 115. The floating weight of the south span was 380 tons, which would give the scows a displacement of 2 ft. 8 in. It happened at this time that the tide did not rise more than 3 ft., and as about 4 in. was lost before the plungers got proper bearing on the sand, the scows failed to float the bridge clear of the platform. The replacing was postponed for 24 hours, which was satisfactory to the Traffic Department, as February 22d was a holiday, and the traffic, therefore, light, and

the time of high water would not change appreciably during the 24 hours.

The scows, which had been removed from under the bridge span to avoid the danger of unexpectedly floating the bridge during the intervening high tide, were again placed in position on Sunday morning, February 21st. As a steady west wind had produced exceptional low ranges of tide, the sand boxes were increased 12 in. in height, and jacks were used between the bridge span and the scows to produce an initial displacement of the scows of 8 in. at low tide. The south span was floated successfully at this time, 2 hours before high tide, with 9 in. of tide to spare, although the range of tide was only 2 ft. 11 in. When the span was clear of the platform, the base of rail was 36 ft. 7 in. above the water level at the south end of the span, and 35 ft. 10 in. at the north end. The displacement of the scow at the south end was 2 ft. 6 in. and at the north end 2 ft. 10 in.

As the scows gradually cleared the platform, they were braced together by timbers cut in advance for this purpose, so as to insure the scows remaining in their relative position during the towing. The bridge was towed to the Company's dock and was tied up there to await the next tide, when it was to be placed.

In the mean time two other scows had been provided with timber trusses similar to those on the scows carrying the new bridge span, and, at 10 P. M., when the next low tide occurred, they were towed to and placed under the old south bridge span. Blocking was placed between the timber trusses and the under side of the old bridge, and at 1.30 A. M., the blocking was permitted to bear. The last train passed over the bridge at 1.58 A. M., and all the rail connections between the bridge span and the structure on the shore were removed by 2.07 A. M. At the same time, the swing span was opened so as not to interfere with the work.

At 3.30 A. M., the old span was clear of the supporting piers, except at the northeast corner, and as the tide was approaching its highest level, the derrick-boat, which had been used for erecting the bridge, was used to assist the tide in getting the span clear of the piers at this point. The old span floated clear at 3.58 A. M. on top of the tide, and was towed to the north and afterward returned through the open swing span to the Company's dock.

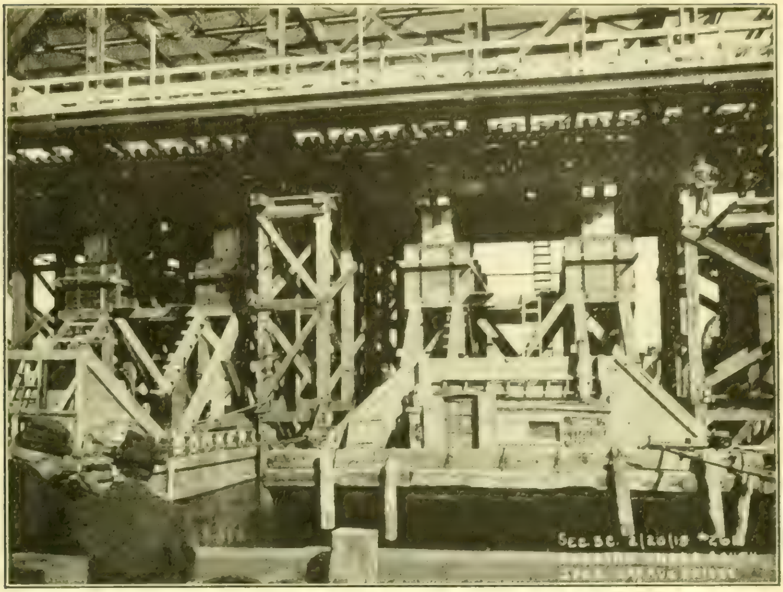


FIG. 115.—SAND JACKS.

FIG. 116.—HARLEM RIVER BRIDGE DURING THE CHANGING OF THE SWING SPAN.

Immediately thereafter, the new bridge seats and expansion rollers were set on the piers by small derricks, and the new span was floated into position by 4.30 A. M., 3 ft. 6 in. above its proper elevation. The tugboats maneuvered the bridge very closely to its proper location, and it was then secured to the piers by two crossed sets of blocks and falls at each end and by steamboat ratchets to control longitudinal movements.

The tide was now falling, and the settling of the bridge was assisted by letting sand flow out of the sand-jacks. The position of the bridge span was kept in continuous adjustment, and a careful final adjustment was made immediately before the bridge was about to bear on its seats, by inserting a bar between the ropes of the falls and twisting the ropes until the bridge was accurately centered.

The new south span was brought to a bearing at 6.10 A. M. The track connections were then made, and the swing span closed, and at 6.56 A. M., 4 hours and 58 min. after traffic had been suspended, the first train passed over the new bridge. The tide continued to fall, and at 7.40 A. M., the scows cleared the bridge and were towed away, taking all the falsework with them.

The next favorable tide for setting a bridge span occurred on Sunday morning, March 6th, 1915. The day before the new north span had been floated off the platform, the range of tide was nearly twice as great as when the south span was floated. The weather at this time was very unfavorable. A heavy northeast snowstorm, with high winds, commenced on Saturday morning and continued throughout the day and the following night, while the bridge was being placed. It had been arranged that traffic should be suspended at 1 A. M., and the scows which were to remove the old north span were brought to a bearing under the bridge some time prior to this. On account of the northeast wind, however, the tide rose with great rapidity and was so high that traffic had to be suspended at 12.44 A. M. The track connections were immediately removed, but with some difficulty, as the tide was already lifting the bridge. When clear of the piers, the old north span was towed away to the north and returned through the open swing span to the Company's dock.

The new north span was towed into position at 12.35 A. M. and was brought to a bearing on the piers at 4.31 A. M. by using the sand-jacks, while the tide was still 1.0 ft. above mean high water.

The erection of the new swing span commenced on March 10th, 1915, and the placing of the new operating machinery was started on April 5th. As described previously under the heading "Details of Design", the machinery first erected did not work properly, on account of the flexibility in the support of the bearings of the beveled gears. New castings, with bearings for both the gear wheels made in the same casting, therefore, were designed and fabricated, and the erection of the machinery was not finally completed until August 15th, 1915.

In the mean time the necessary preparations on the existing center pier for carrying the new swing span had been in progress. As stated, the new swing span was center bearing, and the old one was drum bearing. In order to provide space for the center casting, it was necessary to excavate a pit in the center of the pier, 8 ft. in diameter and 4 ft. 3 in. deep, directly under the center pivot of the old swing span. A frame of I-beams, therefore, was placed on top of the pier to support the pivot, and was held in position securely by timber struts wedged in between the steel frame and recesses in the pier made for the purpose. The excavation was then started, by what amounted to sinking a shaft into the pier outside of the center pivot, and, from that, the pit for the new center casting was tunneled out. The work was greatly hampered by lack of head-room, having to be done, until sufficient depth had been reached, while lying on top of the pier. When the excavation was completed the bottom of the pit was made level with a granolithic finish, and a beveled ring with an inside diameter of 7 ft. was set in place and anchored securely to the pier. In order to insure a uniform bearing, a sheet of lead, $\frac{1}{16}$ in. thick, was placed under the beveled ring. The purpose of this ring was to act as a guide when the center casting was to be set in place, which was to be done simultaneously with the placing of the swing span, so that no time should be lost in instrument work while the traffic was suspended.

In addition to this work, plates were placed on the pier and anchored and grouted to form seats for the new wedge castings, and the rack for the new bridge, which was made in twelve sections, was placed in position and anchored inside of the existing circular track. This track, which had a tapered surface, was maintained temporarily to act as a

track for the balancing wheels of the new bridge, but was removed after the latter had been put in place.

On the rest piers, the existing granite cap-stones were cut to fit the new wedge and latch castings. As the new wedge castings were at the same points as the existing castings, it was necessary to remove these while the seats were being prepared. This was done between 5 P. M. and 9 A. M., during which time the bridge is not opened for river traffic.

Before the new swing span was moved, the lower-deck track had been laid complete, and a temporary switch machine had been placed on the new bridge. A new submarine cable was also laid to the center pier to supply power for operating the new swing bridge.

The day chosen for placing the swing span was Sunday, August 22d, 1915. At 12.35 P. M., on August 21st, the four scows which were to carry the bridge were placed in position under the new swing span on the platform. The plungers came to a bearing on the sand in the boxes at 1.15 P. M. At 5.40 P. M., the tide had lifted the bridge clear of the platform 2 hours before and 2 ft. below high tide. The floating weight of the swing span was 1 100 tons, and the average displacement of the four scows under this load was 3 ft. 6 in. Fig. 117 shows the four scows taking the load of the swing span off the platform. The base of rail, when floating, was 35 ft. 2 in. above water level. The bridge was towed over to the east side of the river, just south of the Harlem River Bridge, and was tied up there until the time of placing it had arrived. On Sunday morning, August 22d, low tide occurred at 1.30 A. M. The two scows which were to float off the old span on the rising tide were placed under that span and were allowed to take bearing at once. Traffic across the bridge was suspended at 1.54 A. M. At 5.30 A. M., the old span was lifted clear of the piers and towed to the south and tied up at the Company's dock. The old spider casting and bearing wheels were left on the center pier when the old span was removed. After cutting the spider in convenient pieces to be handled, all these parts were removed by a derrick-boat, which also set the new center casting and the center pier wedges. At the same time, the latch castings and the wedge castings on the end piers were set by small derricks placed on the upper deck of the end spans. Fig. 116 shows the derrick working at

the center pier, and also shows the old center span removed to the right and the new span waiting to be placed at the left.

The new span was floated into its proper positions over the piers at 7.45 A. M., and was secured to the end spans in proper alignment in a manner similar to that described for the south span. In addition to this, 6 by 8-in. timbers were bolted to the approach spans and acted as guides for the end post of the swing span. The lowering of the bridge was then commenced by the sand-jacks, aided by the falling tide, and, at 10 A. M., the swing span landed in its final position.

The falling tide released the scows at 11.30 A. M., and at 12.43 P. M., when the track and signal work was completed, the first train crossed the new span.

Steel Erection.—Section No. 5-D.—This section extended from the Harlem River through the yard of the New York, New Haven and Hartford Railroad and the Company's yard to 133d Street. Steel erection had been in progress north of this section, and the traveler during this work proceeded directly from the north to this section at 133d Street. In order to place the new structure across 133d Street, the structure had to be shored as some of the supporting columns were moved. Generally speaking, one of the two columns which supported the structure carrying one of the tracks was removed, and the structure was supported temporarily by an **A**-frame, as shown by Fig. 118, braced diagonally to the remaining column. South of 133d Street the tracks were carried directly above a machine shop in the Company's yard. In order to place the supporting columns, which went through the roof and the floor of the shop, and were supported on foundations below the shop floor, the roof was entirely removed. After the structure was erected, the roof was replaced.

The traveler proceeded through the Company's yard and the freight yard of the New York, New Haven and Hartford Railroad, the steel being furnished on flat cars operating on the existing tracks of the elevated structure.

Steel Erection.—Section No. 6-A.—Prior to erecting the new steelwork in this section, which comprises the Third Avenue Elevated Line, from 133d to 147th Streets, through the Company's private right of way, the existing structure had to be moved to make room for the new one. The existing tracks through the right of way were supported on brick piers with granite cap-stones. New concrete piers were built to support the south-bound track at its new location. The north-bound track, including the track stringers, was shifted 6 in. to the east. The

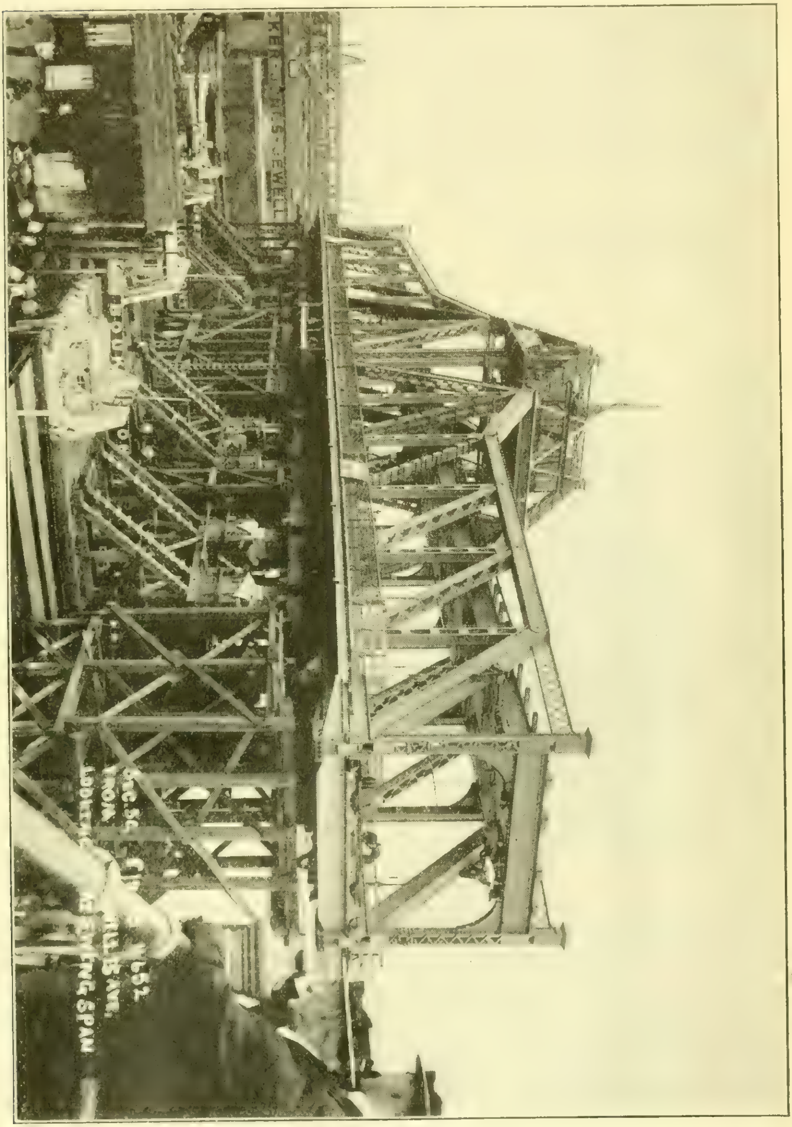


FIG. 117.—SCOWS READY TO LIFT SWING SPAN.

work was done by supporting the track stringers on wooden bents placed close to the piers, one on each side of the pier. The bents consisted of two 12 by 12-in. posts, with cap, sill, and bracing, and with the sill bearing on the foundation of the pier. On top of the bents were placed steel plates on which the girders were to slide. The stringers and the granite cap-stones, to which the stringers were bolted, were jacked up about 1 in., and the shoring bents were then wedged up so as to make the stringers bear on the bents. The shifting of the

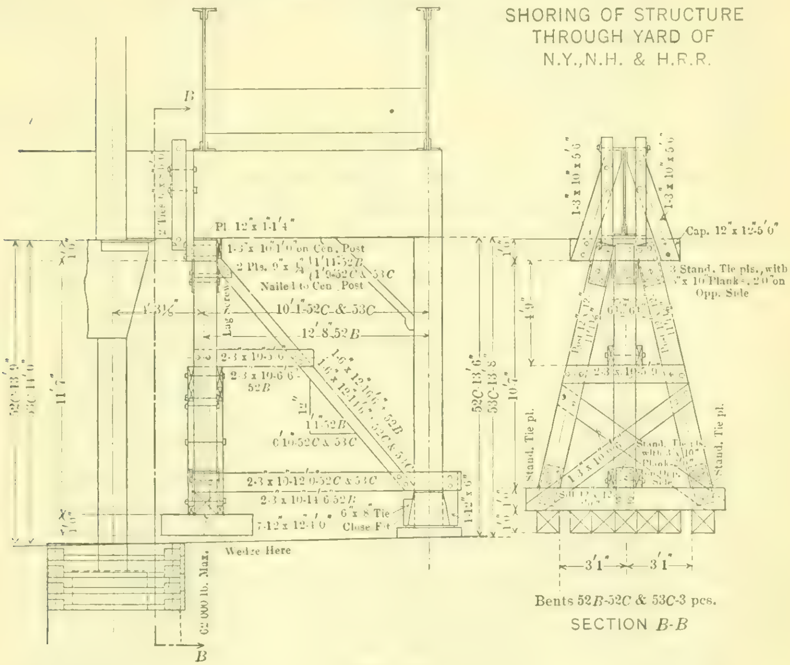


FIG. 118.

stringers was done with steamboat ratchets, as will be described later. When the girders were moved to their proper location, they were lowered with the cap-stone to the new location on the pier, and a new brick extension, supported on the base of the pier, was added to support the overhang of the latter. The extent of the move of the north-bound track was about 750 ft. between 138th and 141st Streets.

The move of the south-bound track was considerably more extensive, both in length and distance. The track was moved between 133d and 144th Streets, a distance of 2 760 ft., and the sidewise movement was

from 12 to 13 ft. The track stringers were shifted from their existing piers to the new piers, which were built in advance of this work, by sliding them over a track, as shown by Fig. 119. Between the piers, and on top of the new pier, the track consisted of rails, supported on the pier on 6 by 8-in. ties and between the piers on timber bents or blocking, as shown. On the old piers, plates $\frac{3}{4}$ in. thick were inserted between the stringers and the top of the piers, on which the stringers were to slide during their moving. The apparatus used for shifting the stringers was steamboat ratchets, hooked to the bottom flange of the far stringers at one end and tied around the new pier at the other end, as shown by Fig. 120.

At the street crossings, where the existing structure rested on steel bents, the structure was supported for moving, as shown in the lower part of Plate L. On top of the shoring girders (4 ft. deep) which had been used for the foundation work, was placed a four-post framed timber tower supporting the stringers to be moved. The existing columns and cross-girders which were connected to the stringers, were moved with the stringers, and a trench was dug in the street so that the base of the columns could move without obstruction. The shoring frame was provided with shoes of steel plates to slide on top of the shoring girders. Steamboat ratchets, connected to the stringers and the shoring, as shown by Fig. 121, were also used for moving these bents. The moving was started at the north end, where there is a curve in the track. About sixteen sets of ratchets (two for each pier or bent) were used. These ratchets were placed on sixteen consecutive piers. Four sets at a time were worked for a move of 3 in., then the next four, and so on, until the track had been moved 3 in. at all sixteen piers. Then another move was made at all piers, except the end pier, then another, leaving the track at the two end piers unmoved, and so on until the track had been moved a distance varying from 3 in. at one end to 4 ft. at the other, which was in the curve at the north end of the entire section, where the move was taken care of by making the curve sharper. After this initial move had been made, the last ratchet was moved to the pier ahead of the first, and the section in hand was moved another 3 in., then another ratchet was moved, and another 3-in. move was made, and so on until the entire track was moved 4 ft., except at the southerly sixteen piers, where the length of the move varied down to 3 in.

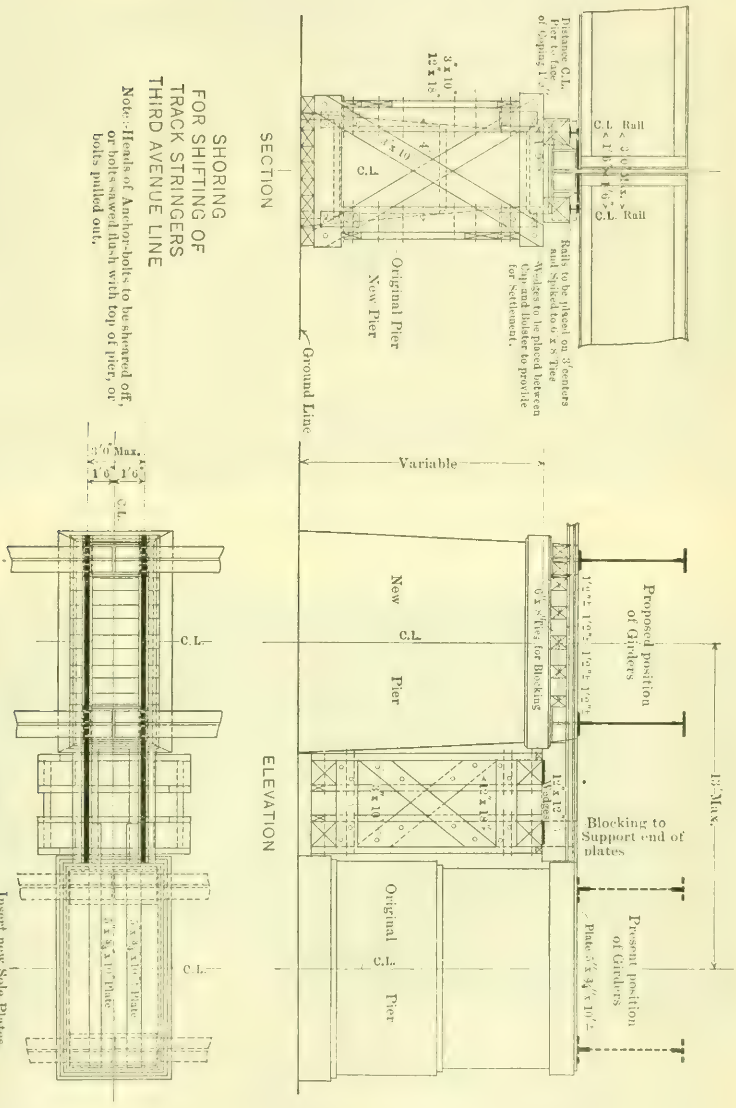

SHORING
FOR SHIFTING OF
TRACK STRINGERS
THIRD AVENUE LINE
Note:–Heads of Anchor-bolts to be sheared off, or bolts sawed flush with top of pier, or bolts pulled out.
SECTION
ELEVATION
PLAN
SCHEME 3
C.L. Rail
C.L. Rail
Rails to be placed on 3′ centers and Spiked to 6″x 8″ Ties
Wedges to be placed between Cap and Bolster to provide for Settlement.
Original Pier
New Pier
Ground Line
Variable
Proposed position of Girders
6″x 8″ Ties for Blocking
New Pier
Original Pier
C.L.
12″x 12″ Wedges
Blocking to Support end of plates
Present position of Girders
13′ Max.
3′0″ Max.
1′6″ 1′6″
Insert new Sole Plates as soon as Girder clears Plates.

FIG. 119.

Then, two more 4-ft. moves were made in the north end, in a similar manner, bringing the track in final position past the 143d Street Station, from which point the length of the move varied down to 3 in. at 138th Street, where the work had to be interrupted, waiting for the completion of the foundations. When these were completed, the track shifting was resumed, and the remaining part was finished in the same manner. The rate of progress was about 4 ft. for a length of 900 ft. per day.

On the portion of the line on which the south-bound track was moved, there were three stations with island platforms. To maintain the service of these stations while the track was being moved, it was necessary to widen the platforms to conform to the location of the track at each stage of the shifting. Extension platforms were provided, therefore, in the following manner. Between the wooden joists of the platform were placed loose additional joists resting on top of the platform stringers. At the end, where the platform was to be extended, these joists were connected to uprights supported on the track ties. This work could be done without interfering with the trains. As the track structure was moved away from the platforms, the joists were pulled out with the ties and planked over immediately.

The length of the track in its new position was more than 1 ft. less than in its original position. It was necessary, therefore, to offset the ends of the track stringers in two adjacent spans at a number of places, so as to permit the girders to slide past each other. To take up the shortening of the rails, two expansion rail joints were introduced in the track.

The steel for the new structure was erected with a traveler, starting at the north end of the section. The material was usually trucked to the intersecting streets, where it was unloaded and moved on rollers into the right of way, where the traveler picked it up.

Steel Erection.—Section No. 6-C.—The erection work on this section, which included the Third Avenue Line from 147th Street to Fordham Road, comprised generally the replacement of existing columns with new ones, the erection of center-track stringers, and the remodeling of the structure at the stations.

Altogether, 699 columns were replaced on this section. While the work was being done, the cross-girders were shored, as shown by Fig. 122. Prior to placing the shoring, the rivets connecting the columns to

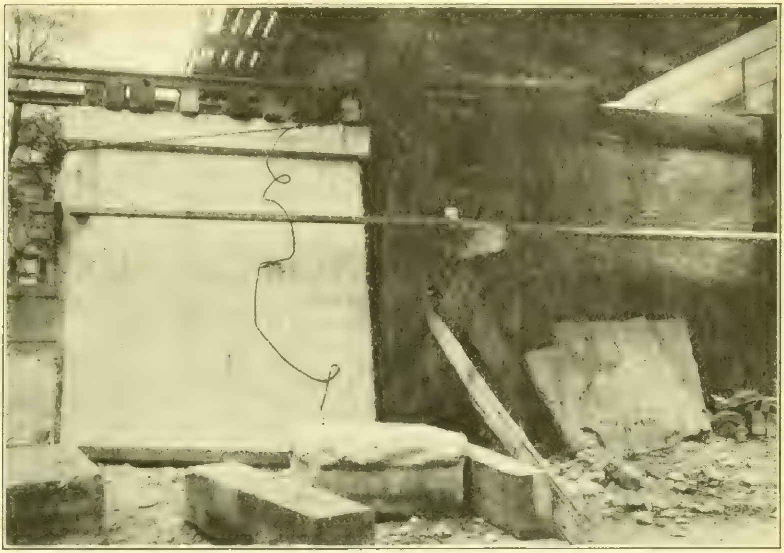


Fig. 120.—Shifting Apparatus on Pier 75.

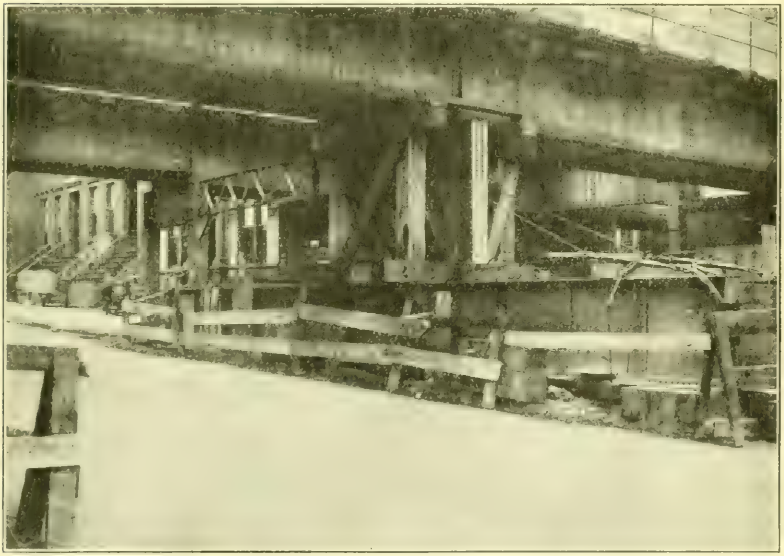


Fig. 121.—Shifting Apparatus on Bent 73.

the cross-girder were cut out, and new stiffeners were provided on the cross-girder. The shoring was then placed, and the cross-girder was jacked up sufficiently to transfer the load to the shoring and to get room to remove the old column and place the new one.

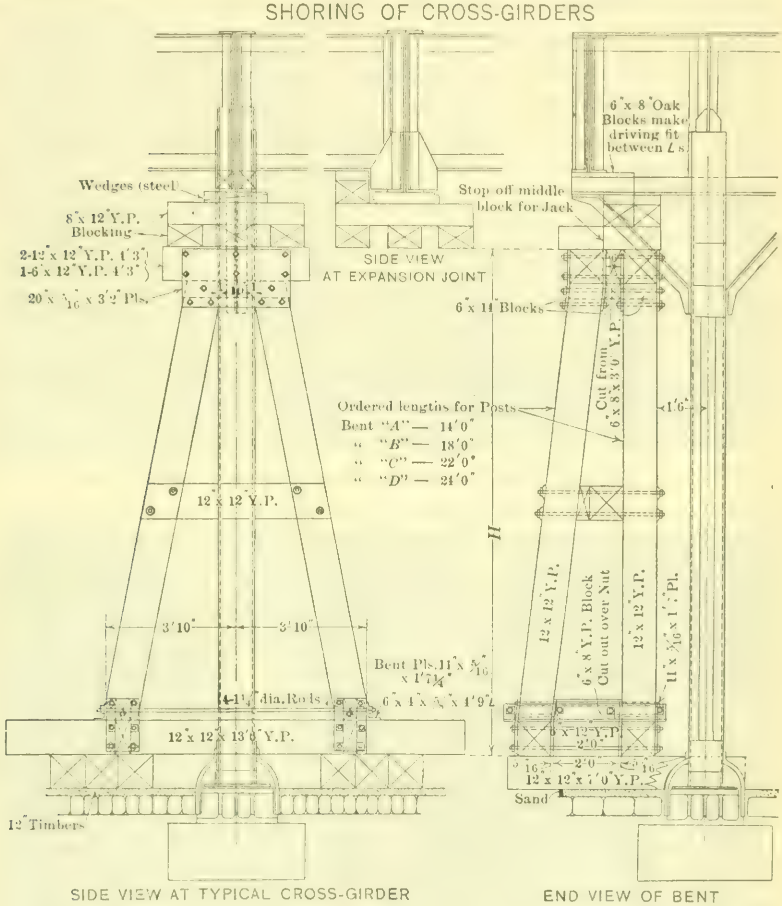


FIG. 122.

In conjunction with the placing of the new columns, the work of raising the entire structure was carried out at stations, where the new arrangement provided a mezzanine station below the track structure. Generally, the head-room under the structure was not sufficient for the mezzanine, and, in some cases, the structure had to be raised 19 in. The work was done by first placing the shoring previously described for replacing the columns under all the cross-girders to be

raised at a station; then the rivets in the end-connection angles of the track stringers were cut out and replaced by bolts, and an additional bearing for the stringers was provided by wedging oak blocks between the bottom flanges of the stringers and the cross-girders. At the same time, the old columns were cut loose from their cast-iron bases. The structure was then jacked up 2 in. at a time, and the raising of the structure was followed up by blocking and wedges on the shoring, as well as between the bottom of the columns and their cast-iron bases, so as to provide double security against possible settlement during the jacking operations.

Prior to the erection of the new center-track stringers, the new seats supporting these stringers were erected from scaffolds hung from the structure. The stringers were erected with a traveler between the south end of the section and 176th Street. North of this point the derrick-car, which, up to this time, had been used on other sections, was used to erect the stringers. The traveler started at the south end of the work and moved north.

The girders erected by the derrick-car were trucked to Westchester Avenue, just east of Third Avenue, through which street there is an elevated railway connection between the Third Avenue Line and the elevated portion of the existing subway line. As this connecting line is not used for traffic between 9 A. M. and 4 P. M., the hoisting of the girders from the street to the structure could be done there without interference with traffic. The method of procedure was as follows: The train, consisting of the derrick-car, a flat car, and a motor-car, was run in on this connecting branch, and two girders were hoisted and placed on the flat car. The train then proceeded north on the Third Avenue Line immediately behind a passenger train. When it arrived at the point where the girders were to be erected, the derrick-car was blocked and guyed, and one of the girders was erected. If there was time to erect the other girder without holding up the next train behind, it was done, otherwise the derrick-car train was started after erecting the first girder, proceeded to a cross-over, returned on the south-bound track and erected the second girder. In either case, the train returned on the south-bound track to the branch in Westchester Avenue for a new load. Fig. 123 shows the hoisting of the stringers at Westchester Avenue. It was found that the cost of erecting the steel with the traveler was nearly 50% higher than with the

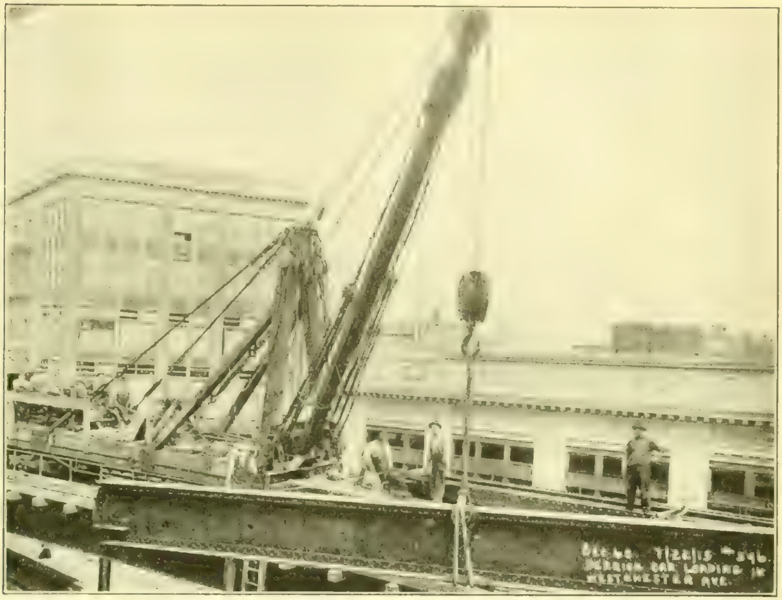


FIG. 123.—ERECTING STEEL WITH DERRICK CAR.

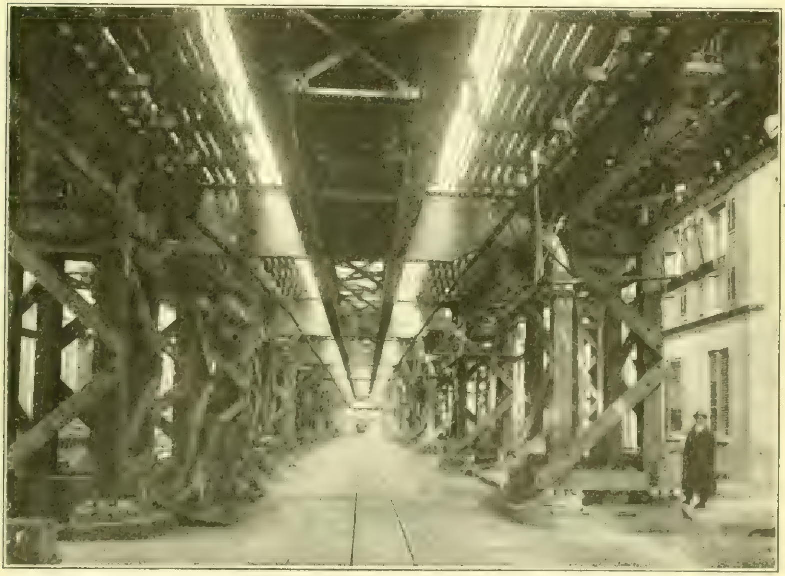


FIG. 124.—STRUCTURE IN GREENWICH STREET.

derrick-car, and that the work could proceed nearly twice as fast with the latter.

The stations at 149th Street, 177th Street, and Fordham Road were reconstructed as express stations. Prior to the reconstruction work each had a center platform with two adjacent tracks, and, in addition, the station at 149th Street had a side platform on the east side of the east track, used for unloading the trains. The new express stations were constructed with two island platforms, with the center track between the platforms. In order to maintain the traffic, the work was carried out in the following manner: First, the new extensions to the cross-girders and the new outside track stringers were erected and the new tracks laid on top of these stringers. At 149th Street, where there was a platform on the east side of the structure, all this work had to be done under the platform without interfering with its use. The next thing to do was to extend the center platform over the existing running tracks, so as to be able to transfer the traffic from those tracks to the new ones. This was accomplished by building temporary platforms of the proper width on top of the new outside tracks, which, when completed, could be shifted over on top of the old tracks adjacent to the existing center platform, thus forming a new center platform of more than twice the width of the old one. The platforms were shifted, one at a time, during the early morning hours, when the traffic was so light that trains could run on a single track past the station, and was generally completed in about 1 hour.

All other stations between 149th Street and Fordham Road were local stations, but, as they had center platforms (with the exception of the station at 180th Street) which were in the way of the new center track, they were all reconstructed so as to provide side platforms to take the place of the existing center platforms. The cross-girders were extended, the extensions being erected with gin-poles, although the new platform girders were generally erected with the derrick-car.

Steel Erection.—Section No. 7.—The work on this section consisted of the reconstruction of the stations of the Ninth Avenue Line at 66th, 116th, 125th, and 145th Streets for express service.

The first station to be reconstructed was that at 116th Street. The additional column bracing was erected by placing light scaffolding around the columns and reinforcing the cross-trusses from hanging scaffolds. When this was completed, the erection of the new outside

track stringers was commenced. As the structure at this point is very high, and as a large number of stringers had to be erected, the usual method of erecting with gin-poles could not be used satisfactorily, and a derrick-car was built especially for the purpose. This car proved so useful that it was afterward used to erect steelwork at other points where conditions were favorable.

The new stringers were trucked to points in the street north of the station where the handling did not interfere with traffic in the street. The derrick-car, standing on the center track, which was not used for traffic, hoisted the girders, one at a time, through the space between the center and the south-bound tracks, placing them on the flatcar in front of it. When a girder was loaded on the flatcar, the construction train ran in on the running track immediately behind a passenger train, to the point where the girder was to be placed. The derrick-car was then blocked and guyed, the girder was placed, and the derrick-car continued south and took the cross-over to the center track, leaving the running track clear. The interval between the running trains was 3 min., and the work of placing the track stringers was completed so rapidly, that generally no delay was caused to passenger traffic. After having arrived on the center track south of the station, the train returned on the north-bound track to the center track north of the station, where another girder was picked up, and this continued until all the girders on the west side were erected. The girders on the east side of the structure were erected in a similar manner, except that the work train was loaded at the point and then ran directly over the south-bound track to the center track south of the station, and there awaited a favorable time to run in on the north-bound track to place the track stringers.

The new north-bound track in front of the north platform was then laid and connected to the existing local track, and the existing north platform was closed to traffic temporarily until the reconstruction of the platform was completed. The new north-bound track was supported on the new line of track stringers and the former outside track stringers of the abandoned local track. The inside track stringers, which no longer carried any load, were disconnected from the outside stringers, and those in front of the old platform were used for track stringers on the west side of the structure; the remaining ones were moved and used as platform stringers for the extension of the platforms.

In the meantime, the platform had been repaired and braced, and was then shifted bodily eastward to its new position. The shifting of the platform was done with jacks and steamboat ratchets. The track stringers removed from the east side were then placed to carry the east rail of the new center track, the west rail being carried by the existing east stringer of the old south-bound track. The center track past the platform was then laid, and temporary connections were made at both ends so that this track could be used for south-bound local service. Traffic was then transferred from the south platform to the new north platform, and the construction operation repeated. When this work had been completed, all the permanent track connections were made and both platforms were used, the north platform for north-bound traffic and the south platform for the south-bound traffic. The new mezzanine station was constructed in conjunction with the work on the track level.

The work on the station at 125th Street, in all general details, was similar to that at the 116th Street Station, the main difference being that the existing station buildings at the ends of the platforms were retained, instead of constructing a new mezzanine station under the structure, and that the cross-girders had to be provided with short extensions at each end to make them long enough to support the new lines of outside track stringers.

The stations at 66th and 145th Streets were of the hump type, and were erected in a manner similar to other hump stations. At 66th Street all work that could be carried out without interference with the center track was first completed, and the reconstruction of the center track was delayed, in order to have it carried out simultaneously with that of the center track at other points south of this station, so as to make the interference with the existing express service as small as possible. When the reconstruction was commenced, it was carried out and completed with an interruption of the express train service of only 2 weeks. The derrick-car was used to erect the towers supporting the new elevated center track, and a traveler erected the remainder of the structure. Two working shifts were used for a short time. The men commenced work at 3.30 A. M. and stopped at 8.20 P. M., in order to insure its completion within the prescribed time.

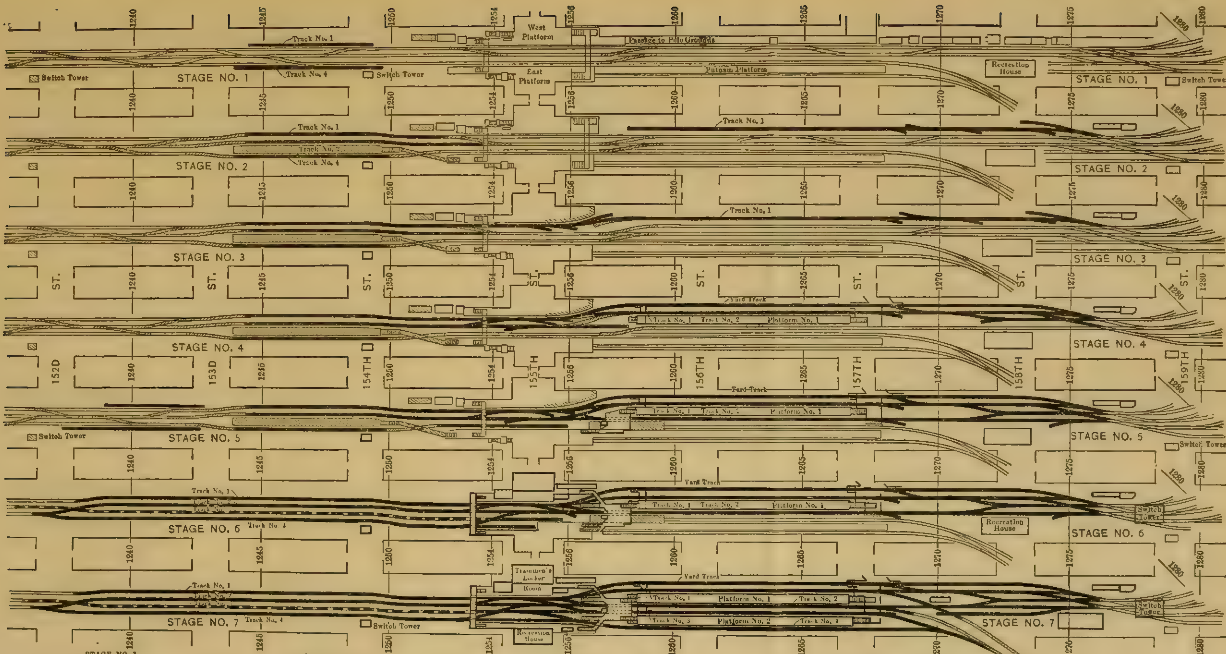
Steel Erection.—Section No. 8-A.—Section No. 8-A extended in Greenwich Street from south of Cortlandt Street to 9th Street, and the

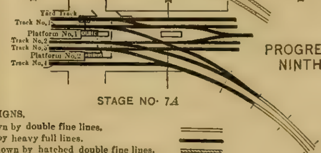
erection work consisted mainly of adding new cross-girders and center track stringers to the existing structure. Fig. 124 shows a portion of the existing structure immediately after the erection of the new cross-girders and center-track stringers. It shows the shoring of the outside track stringers, which consisted of four-post framed towers built around the existing columns, where the columns were not removed, and of two similar towers, one at each side of the existing column, where the columns were replaced with new ones. The columns were generally removed at the new express stations, where the additional load of the express platform required heavier ones, and also where the existing columns were of the type shown to the left on Fig. 124. This, which was the original type of columns used on the elevated lines, consisted of four 6-in. I-beams tied together at intervals. The base of the column was an iron casting, into which the I-beams of the columns fitted and were held by rust caulking. Fig. 124 shows the base casting and a part of the column shaft, which was burnt in two to facilitate removal; it also shows the original track stringers on the west side of the structure, which were only about 24 in. deep, and, in the course of time, had been reinforced with another set of 24-in. stringers, placed directly on top of the original girders. These track stringers were removed after the center track was completed, the south-bound traffic being diverted temporarily to the center track.

At the stations which were converted for express service, the north-bound platforms and track were shifted a few feet east to make room for the additional track and platform. This was accomplished by supporting the ends of the track stringers to be moved on six-post shoring towers, wide enough to carry the stringers both in the original and the new position. As most of the cross-bents were on a skew, it was generally necessary either to cut off a portion of the track stringers moved or to lengthen them. In all cases the ends of the stringers were remodeled to connect to the new cross-girders or to the existing cross-girders in their new position.

The erection of the new cross-girder and track stringers was done with two travelers, one starting at the north end and the other at the south end of the section.

Steel Erection.—Sections Nos. 8-B and 8-C.—Section No. 8-B comprised the construction of the hump stations at 14th and 34th Streets on the Ninth Avenue Line, and Section No. 8-C comprised the elimina-





STAGE NO· 7A

PROGRESS OF ERECTION NINTH AVENUE LINE

STAGE NO. 1

1.—Track Layout same as at present.
2.—Place Cross-Girder Extensions 1236*E* to 1252*E* and 1238*W* to 1252*W*, removing Sidewalk Col. 1248*E*, and place Outside Track Girders 1236*E* to 1252*E* and 1238*W* to 1252*W*, and Switch Tower at 1249*E*.
3.—Place temporary Cross-overs as indicated.
4.—Lay Portions of Tracks Nos. 1 and 4 as indicated.
5.—Remove Passage Way to Polo Grounds and adjacent Yard Buildings.
6.—Erect temporary Signal Tower at Bent 1236.

STAGE NO. 2

1.—Connect with old Track Portions of Tracks Nos. 1 and 4 now laid and use for Traffic.
2.—Shift old Track Girders on Center Line of Structure, Bents 1247 to 1250, closing Track No. 2 during this Operation.
3.—Lay Portion of Track No. 2 and connect to present Tracks as shown, and use for Traffic.
4.—Construct temporary Platform and Bridge from Bent 1244 to Bent 1252.
5.—Erect new West Sidewalk Columns and Cross-Girder Extensions from Bent 1258 to North End of Work.
6.—Raise present Track Girder at Bent 1270 and lay Track No. 1 north of Bent 1258.

STAGE NO. 3

1.—Use Temporary Platform for South-Bound Traffic.
2.—Remove North foot Bridge and Remodel West Platform at North End.
3.—Raise present Track Girders and erect such new ones as will enable Track No. 1 to be completed from Bent 1252 to 1258, and lay this Portion of Track No. 1.
4.—Place new Yard Track Girders at Bent 1258 and from Bents 1267 to 1274.

STAGE NO. 4

1.—Shift Girders 1257 to 1267 from present West Track to new Yard Track and lay this Track.
2.—Construct Platform No. 1 and Parts of North and South Mezzanine Stations shown and Stairways to Street.
3.—Raise Track No. 2 to new Position from Bents 1253 to 1258.
4.—Place Temporary Cross-over, as indicated.

STAGE NO. 5

1.—Stop Trains at temporary Platform only.
2.—Raise Girders and construct Track No. 2 from Bent 1258 to North End and Track No. 2 from Bents 1252 to 1258.
3.—Place Girders that support upper Station, and construct part of Station shown.
4.—Finish Structure on West Side of Putnam tracks north of Bent 1270 and lay tracks as shown ready for Connections.
5.—Lay Portions of Tracks Nos. 1 and 4, as indicated, ready for Connection.

STAGE NO. 6

1.—Remove temporary Cross-over between Tracks Nos. 1 and 2 in Spans 1256 and 1257, and finish the permanent ones south of same and use Platform No. 1. for all Traffic.
2.—Finish Parts shown of Upper Stations.
3.—Complete Tracks Nos. 1, 4 and 2 south of temporary Platform, abandoning Temporary Cross-overs and Switch Tower.
4.—Remove temporary Platform and East Platform south of Bent 1254 and finish Track No. 3 south of Bent 1250 and Track No. 4 between Bents 1251 and 1254.
5. Reconstruct West Station.

STAGE NO. 7(a)

1.—Reconstruct East Station.
2.—Reinforce East Cross-Girder Extension Bent 1256.
3.—Complete Tracks Nos. 3 and 4 as far North as indicated.
4.—Construct Platform No. 2 and finish Mezzanine Stations.
Stage 7*A*. Complete Tracks as indicated.
a) Not to be started before N.Y.C. Putnam Division's Tracks and Platforms are abandoned.

CONVENTIONAL SIGNS.

Original Tracks shown by double fine lines.
Final Tracks shown by heavy full lines.
Temporary Tracks shown by hatched double fine lines.
Original Station Platforms, Buildings and Bridges shown by fine line outlines.
Final Station Platforms, Buildings and Bridges shown by heavy line outlines.
Temporary Station Platforms, Buildings and Bridges shown by hatched outlines.

tion of the express track grade crossing at 53d Street, where the local tracks of the Sixth Avenue Line connect with those of the Ninth Avenue Line. The erection of the steel for the hump stations was carried out in a similar manner to that described, and needs no further comment.

At 53d Street the approaches were erected in a similar manner to the hump stations, but the approaches were longer, because the head-room required under the new structure at 53d Street, where the south-bound Sixth Avenue track crosses under the new structure for the express track, necessitated a greater height of the express track over the local tracks than that required at the hump stations. On account of the crossings at 53d Street, the span length there had to be made about 99 ft., and the over-head track was carried on a through bridge construction. The erection of the new structure was done with a derrick which, however, was not sufficiently strong to erect the through bridge girders. Another and heavier derrick, therefore, was erected in the street at the west side of the structure, and supported on towers in order to bring it to the level of the structure. Fig. 126 shows one of the girders in course of erection. The traffic on the lines was interrupted during the erection for a period of about 18 min. for each girder. After this span was erected, the large derrick was used to lift the traveler which erected the remainder of the work across the through span, which was too narrow for the traveler to pass through.

As stated when describing the erection of the 66th Street Station, Section No. 7, this station and the work under Sections Nos. 8-B and 8-C were carried out and completed at the same time, so as not to interfere with the express traffic more than necessary. This work, which involved the erection of 3 000 tons of steel and the removal or relaying of 7 000 ft. of track, was accomplished in 2 weeks.

Steel Erection.—Section No. 10-B.—Section No. 10-B comprised the reconstruction of the station at 155th Street and Eighth Avenue to serve the new express track, and the connection to the new rapid transit line in Jerome Avenue.

The erection problem was essentially one of maintaining traffic during the work. The method of doing this was worked out in advance, as shown on Plate LIV, and was followed throughout, as far as it was completed, by the contractors. The final stages of the work were not completed by the contractors, as the east platform could not be built

before the Putnam Division of the New York Central Railroad vacated the space the platform was to occupy, and this again was dependent on other work on the connection, which was in progress.

Plate LIV shows the different stages of the work. Stage No. 1 indicates the existing layout of the platform and track previous to reconstruction, to which were added a number of temporary cross-overs to divert the traffic from the tracks under reconstruction. During this stage, the cross-girder extensions and as many of the new outside track stringers south of the station as could be placed without interfering with traffic on the running tracks were erected, and a new temporary signal tower was built at 152d Street to take the place of the existing tower at the south end of the then existing west platform. This new signal tower was supported on timber bents from the street level. The track was laid as far as possible on the new track stringers south of the station, and, at the same time, the existing passageway from the west platform to the entrance to the Polo Grounds was removed. In the second stage, the newly laid portions of the tracks south of the station (marked No. 1 and No. 4) were connected to the old tracks, and the corresponding portions of the old tracks were abandoned. Such track connections were made without interruption of traffic, as they were generally completed in about 6 min. The existing center track, including the track stringers between 153d and 154th Streets, was then shifted west to its new permanent position (marked Track No. 2), and a new temporary platform was built between Tracks Nos. 2 and 4, connecting to the old east platform with a bridge over the running track. Fig. 127 shows the new platform and bridge in course of construction. At the same time the new columns and cross-girder extensions on the west side of the structure north of the old station were erected, and the new track (Track No. 1) on this portion was laid.

During the next stage (Stage No. 3) the new temporary platform was used in place of the old west platform, which was abandoned for passenger service, as well as the portion of the old west track in front of the platform, and this permitted the completion of Track No. 1 past this platform. During this stage, also, the new track stringers in the northerly stretch, for the new west track (marked Yard Track), were placed.

The remainder of the track stringers for this track were placed during the fourth stage, because they were old stringers, formerly

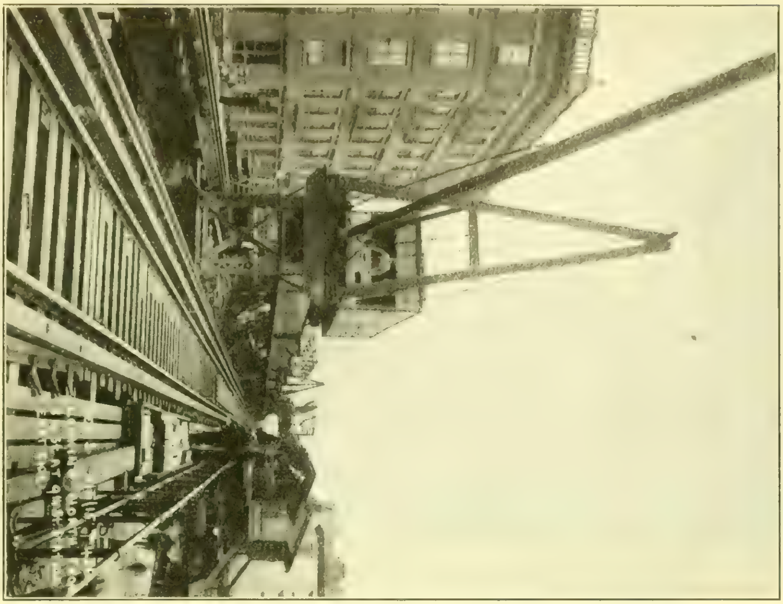


FIG. 125.—STATION ON THIRD AVENUE AT 9TH STREET DURING RECONSTRUCTION.

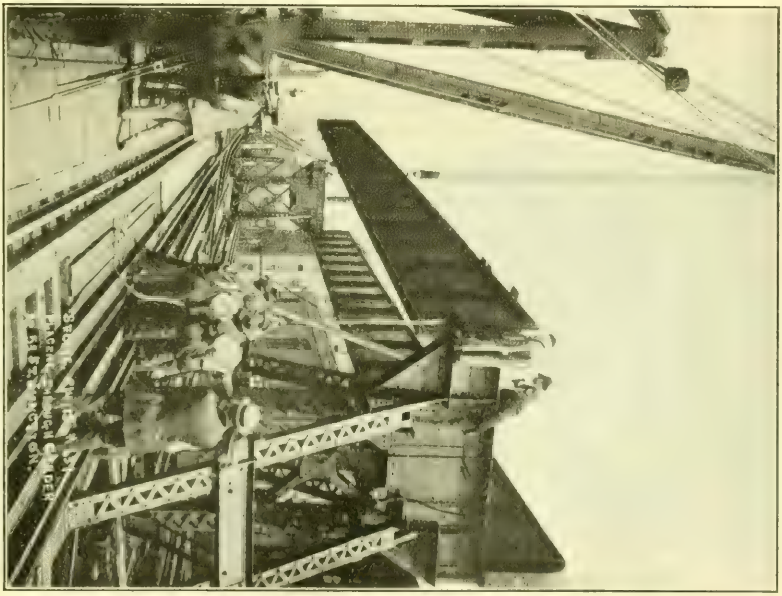


FIG. 126.—CROSSING ON NINTH AVENUE AT 53D STREET.

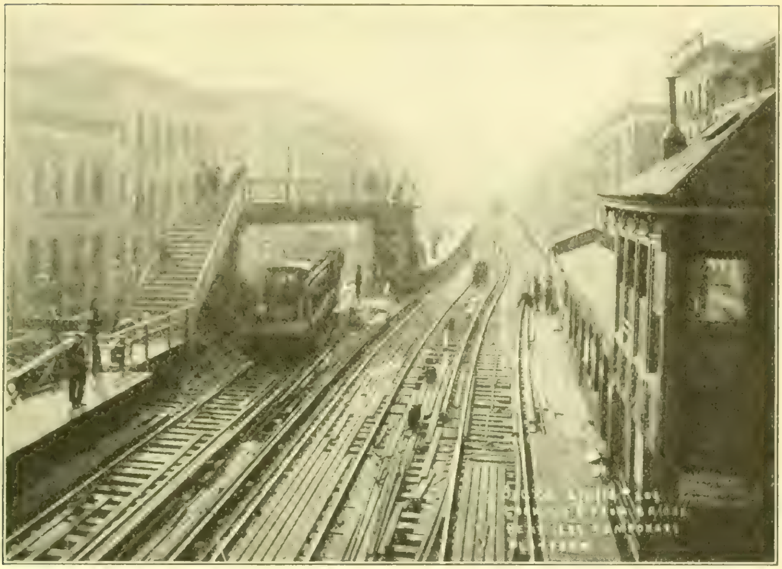


Fig. 127.—Station on Eighth Avenue at 155th Street. Temporary Platform.

Fig. 128.—Station on Eighth Avenue at 155th Street Before Reconstruction.

used under the old west track, which, at this stage, was abandoned and replaced for traffic by the new Track No. 1. The new island platform (Platform No. 1) was then built, and also as much as possible of the new mezzanine stations at the north and south ends of this platform.

At the same time, Track No. 2, in front of the old station, was raised to its new level and connected with a temporary cross-over to Track No. 1, south of the new island platform.

During the next or fifth stage, the temporary platform south of 155th Street only was used for passenger traffic, and the two new tracks on the west side of the street (Track No. 1 and Yard Track), north of 155th Street, were used for connection to the yard at 159th Street. Track No. 2, north of 155th Street, was built to proper grade, and part of the upper station at 155th Street was constructed.

In the sixth stage, the new Platform No. 1 and the mezzanine stations were opened for traffic, and the temporary platform was abandoned, making room for the completion of the work south of 155th Street. The last stage can be completed without interfering with traffic, when the east side of the structure is abandoned by the Putnam Division.

Fig. 128 shows the original layout north of 155th Street. On the left side is shown the north end of the west platform and the passageway to the Polo Grounds; on the east side is shown the Putnam Division tracks and platform. Fig. 129 shows the same view after the passageway had been removed and a platform of the new steel on the west side of the structure had been erected. It shows the new track girders raised above the level of the existing tracks, which was done in order to obtain the necessary head-room for the mezzanine stations. Fig. 130 shows the new platform and upper mezzanine station completed for operation.

The complete reconstruction, as outlined, had to be carried out during the period between baseball seasons, as all the Major League games in New York City were played on the Polo Grounds, and by far the greater portion of the spectators at these games are handled through this station. The baseball season closed on October 20th, 1914, and the next season opened on April 13th, 1915. The reconstruction work started immediately after the close of the season, and

was completed by April 1st, 1915, but only by intense application throughout the whole period.

After this work was completed, the new over-grade express track between 150th and 155th Streets was erected with a traveler, starting at the south end of the incline.

Stations.

When passengers arrive at a station to take a train, they generally come singly or in small parties, and no delay is caused by passing through the station building to the platform. When, on the other hand, passengers are discharged from a train at a station, they are all ready to leave the platform at the same time, and, therefore, the exits to the street should be made as direct as possible. The stations remodeled or rebuilt under the "Manhattan Elevated Improvements" were designed with this principle in view. The exits were generally arranged to be as direct as possible, without any doors to obstruct the traffic.

Fig. 131 shows a diagram of the mezzanine station at Canal Street and the Bowery on the Third Avenue Line. From the four stairs connecting the mezzanine station to the street are passages leading to the ticket office and from there past the ticket boxes to the stairs leading to the platforms. The stairs which connect the mezzanine station with the platforms are used both for access to and exit from the platforms, as it is practically impossible to prevent passengers from using the stairs most convenient to them. Passengers leaving the platforms descend the stairs to the mezzanine station, and leave through the exit openings without interfering with the passengers coming in past the ticket booth. The mezzanine station is provided with toilet rooms, and only these and the ticket office are heated, as it would otherwise be necessary to provide doors, which would interfere with the progress of the passengers.

Fig. 132 is a plan of the mezzanine station at 125th Street on the Second Avenue Line. The general plan of this station is the same as that described, but, as traffic at this point is not so congested, the waiting-room is enclosed and heated. The exit is arranged direct, as at the Canal Street Station. It will be noted in both cases that all the toilet fixtures are backed against the walls of the heater rooms. This is done partly to keep the fixtures from freezing, but mainly to

FIG. 129.—STATION ON EIGHTH AVENUE AT 155TH STREET DURING RECONSTRUCTION.

FIG. 130.—STATION ON EIGHTH AVENUE AT 155TH STREET AFTER RECONSTRUCTION.

keep the pipes away from the toilet-rooms, so that they can be inspected and repaired without working in the toilet-rooms and also to prevent them from being destroyed by vandals. In fact, all the toilet fixtures are arranged so that nothing can by any ordinary means be broken or unscrewed and taken away; the flush-tanks, for example, are

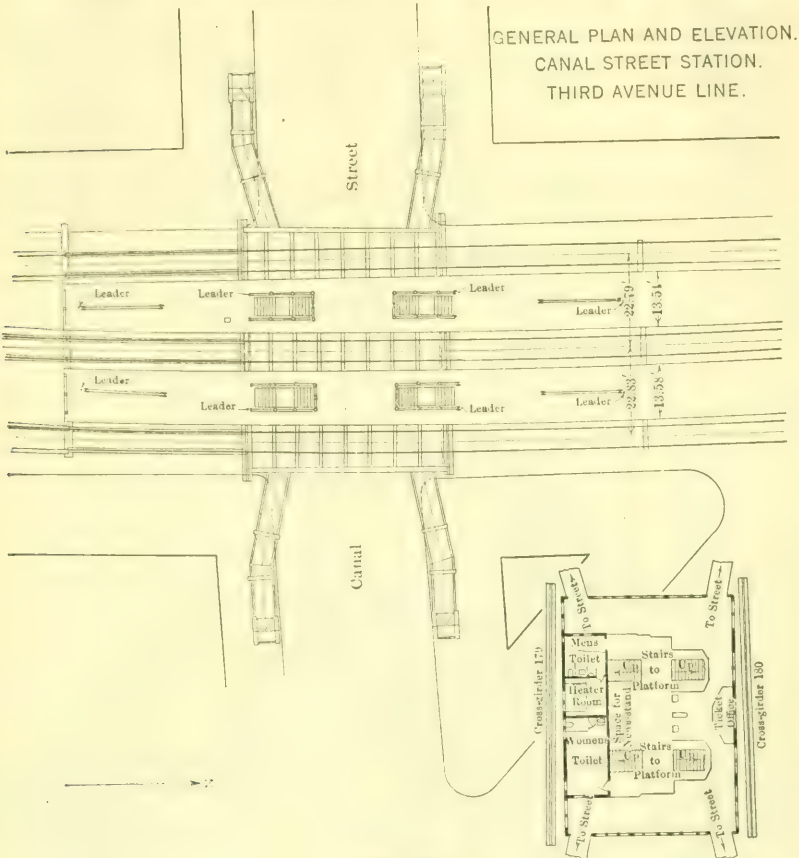
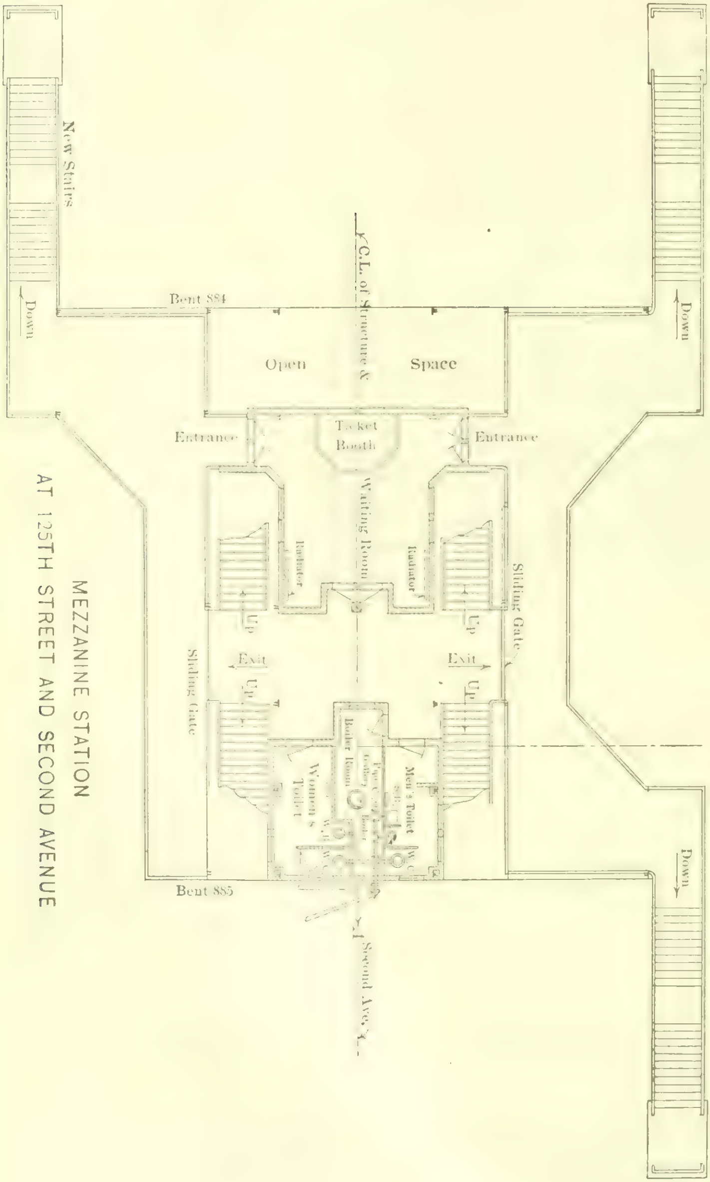


FIG. 131.

FLOOR PLAN OF STATION

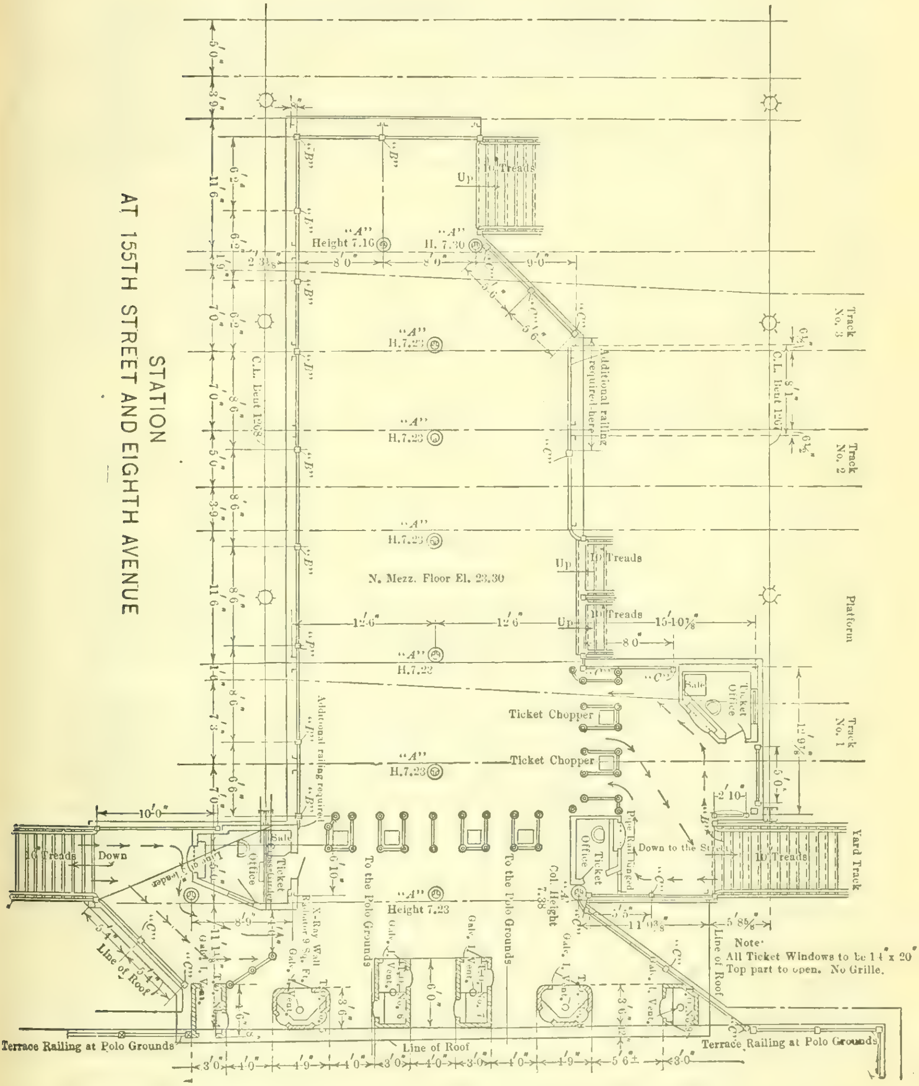
worked by push-buttons set into and fastened at the back of the partitions.

The station at 155th Street and Eighth Avenue, on the Ninth Avenue Line, is provided with two station buildings at the south end of the platforms and with one at the north end. At the south end, one station building is below the track structure, and serves pas-



MEZZANINE STATION
AT 125TH STREET AND SECOND AVENUE

FIG. 132.



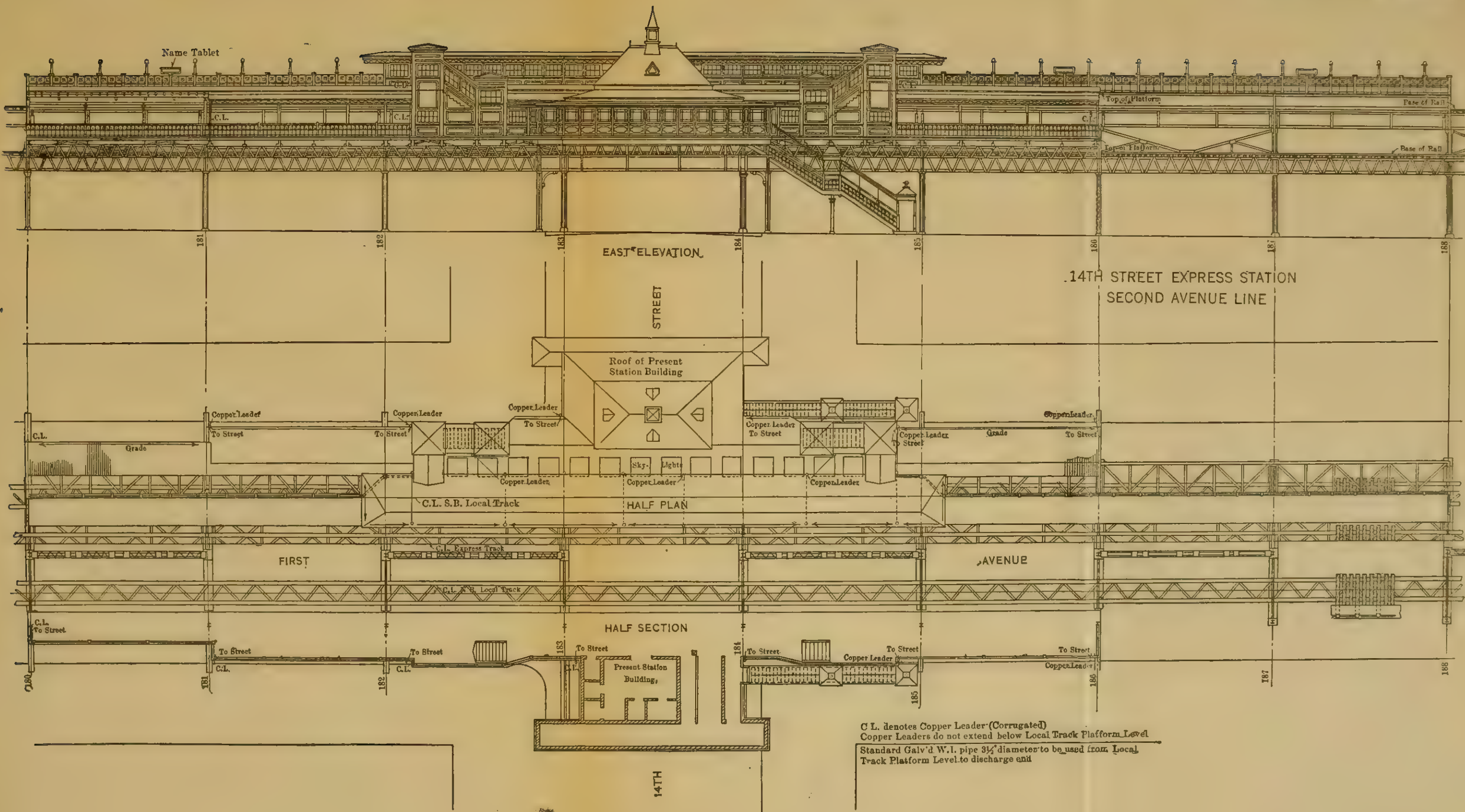
STATION
AT 155TH STREET AND EIGHTH AVENUE

FIG. 133.

sengers from Eighth Avenue; the other station building is above the track structure, and is connected with the viaduct forming the approach to the McComb Dam Bridge across the Harlem River. The station at the north end of the platform is a mezzanine, and is mainly provided to give direct access to the baseball park at the Polo Grounds. Fig. 133 shows the arrangement of this mezzanine station, which was built to handle a large number of incoming passengers at the same time. The main floor of the baseball park building leading to the grand-stands is at the same level as the mezzanine station and connected directly by a wide passage, on which a large number of ticket sellers and ticket choppers can be placed. The stairs leading from the mezzanine station to the platforms are as wide as the width of the platforms and the track layout would permit. During the last season, when only the west platform was constructed, the traffic was handled without serious congestion, and when both platforms are completed, the facilities will be doubled.

Plate LV shows a typical arrangement of a "hump" station. The existing station buildings contained all the necessary facilities, and no addition was made to them. A passenger entering the station would, after buying and depositing his ticket, pass out on the existing local platform. If he desired to take a train on the upper level, he would ascend the stairs leading to the upper platforms. These stairs are placed in such a manner that they are close to the exits and, therefore, give direct access to the street.

The platform construction consists of platform girders on top of which are placed I-beams, carrying the wooden floor-joists. Generally, the floor-joists were set on top of the I-beams, but when it was necessary to make the floor construction shallow, these joists were framed into the I-beams and supported on angles riveted to the web of the latter. The floor of the platforms consisted of 2 by 6-in. planks laid transversely to the length of the platform. As the lumber used for the planks was not well seasoned, the planks were laid with close joints. After they had been in place for some time and dried out, the joints opened about ⅛ in. The platforms were covered with canopies for either part of their length or their full length, according to the traffic conditions and the location of the station buildings. The canopies were supported by posts. On side platforms usually two rows of posts were used, one being at the back of the platform and


Name Tablet
EAST ELEVATION
14TH STREET EXPRESS STATION
SECOND AVENUE LINE
STREET
Roof of Present Station Building
Copper Leader
To Street
Grade
Sky-Lights
C.L. S.B. Local Track
HALF PLAN
C.L. Express Track
FIRST
AVENUE
HALF SECTION
Present Station Building
14TH
C.L. denotes Copper Leader (Corrugated)
Copper Leaders do not extend below Local Track Platform Level
Standard Galv'd W.I. pipe 3½" diameter to be used from Local Track Platform Level to discharge end

incorporated in the railing. The other row of columns was placed within a few feet of the front edge of the platform, far enough from the edge to make it safe to walk between the post and a train pulling in at the platform. On the center platforms, the canopy was usually supported by a single row of columns, set in the center so as to leave the platform as clear as possible of obstructions. The canopy was covered with tinned iron, laid on tongued and grooved boards.

The portion of the side platforms covered by canopies was usually protected with wind shields at the back of the platform. The wind shields were provided with detachable sash, so that the sash could be removed in the summer or when being cleaned.

The new station buildings were generally covered on the outside with sheet iron and painted, but, in some cases, on the down-town stations, copper was used. The inside woodwork trim was usually comb-grained yellow pine, producing a very pleasing effect.

Track Work.

Throughout the reconstruction work the standard Manhattan Railway track construction (Fig. 134) was used. The specifications for track timber were as follows:

"1.—The timber to be of long-leaved, first-growth, Florida, Georgia, or Alabama yellow pine, straight, square-edged, free from shakes, loose, large, or rotten knots, and every other material imperfection, planed on all sides, and of the full schedule dimensions after planing.

"2.—In no case will any stick be accepted with less than three heart corners, or with more than 1 in. of sap on the fourth corner, or more than 2½ in. of sap on either side at either end of the stick.

"3.—The timber to be delivered, as required by this Company, under the foregoing requirements, irrespective of trade usage or conventional specifications, and to be subject to the inspection and acceptance or rejection of the Company's authorized inspector."

The standard ties are 6 by 8 in. in cross-section, full dimensions, and are 8 ft. long. For the outside tracks, where a footwalk is required, every third tie is 12 ft. long and carries the footwalk. The standard spacing of the ties is 18 in. from center to center. The ties are fastened to the flanges of the track stringers with hook-bolts. The outside footwalk, laid on the 12-ft. ties, consists of five pieces of slatting, 2 by 6 in., full dimensions. Where there is a center track, the spaces between this track and the outside tracks are bridged over

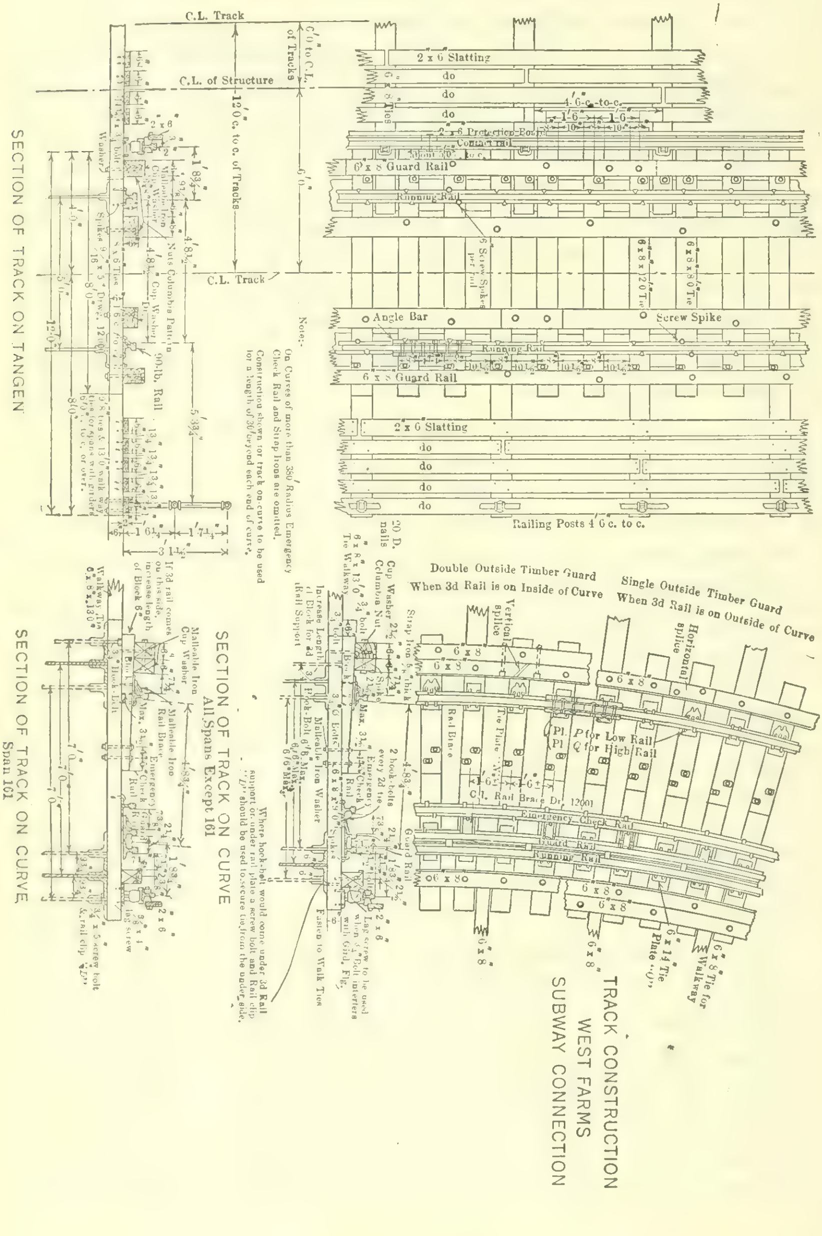

SECTION OF TRACK ON TANGENT
SECTION OF TRACK ON CURVE
All Spans Except 161
SECTION OF TRACK ON CURVE
Span 161
TRACK CONSTRUCTION
WEST FARMS
SUBWAY CONNECTION
C.L. Track
C.L. of Structure
12'0" c. to c. of Tracks
2 x 6 Slatting
6" x 8" Guard Rail
Running Rail
Angle Bar
Screw Spike
6 x 8 Guard Rail
Railing Posts 4' 6" c. to c.
90 lb. Rail
Double Outside Timber Guard
When 3d Rail is on Inside of Curve
Single Outside Timber Guard
When 3d Rail is on Outside of Curve
Emergency Check Rail
Guard Rail
Running Rail
Note:- On Curves of more than 350' Radius Emergency Check Rail and Strap Irons are omitted. Construction shown for track on curve to be used for a length of 30' beyond each end of curve.

FIG. 134.

with standard ties, 4 ft. 6 in. from center to center, and provided with footwalks consisting of four pieces of slatting. The outside tracks are provided with four guard-rails of 6 by 8-in. timber, as shown, to prevent a train from falling off the structure in case of derailment. The standard track rails weigh 90 lb. per yd., have a 5-in. base, and are 5 in. high. On curved tracks the outer rail is super-elevated by elevation blocks, as shown by Fig. 134, and steel guard and check rails are provided. The contact rail, which weighs 100 lb. per yd., is supported on the footwalk ties. The outside footwalks are protected by pipe railings with posts 4 ft. 6 in. from center to center.

The track-laying could generally be carried out without interference with the traffic. Where track was replaced under trains in operation, the work was carried out by the Company's Maintenance of Way Department, which was accustomed to work of this kind.

Plant.

Yards.—The Company's material yard at 128th Street and the Harlem River, which, with the additional land and dock front leased for the use of the "Manhattan Elevated Improvements," had an area of about 50 000 sq. ft., and a dock frontage on the Harlem River of about 700 ft., was the main receiving and distribution depot for material used on the work. Most of the new steelwork for the up-town sections and the greater part of all rails, ties, and other track material, and also most of the scrap material from the structures removed, was handled through this yard. The equipment consisted of a derrick with a capacity of 10 tons, which was used for unloading steel from the lighters and loading it on the trucks, and a traveling derrick, running on rails on the surface of the yard, which was used mostly to handle the track timber. An existing derrick set on the elevated structure adjacent to the yard was used for handling the track material, which was distributed mostly with work trains. The steel was usually trucked to the working site and there picked up by the traveler erecting the structure. The yard was further furnished with a saw-mill and with the necessary storerooms and offices for the contractors and engineers.

Another, but smaller, material yard, on the Company's property at 133d Street, adjacent to the Third Avenue Line structure, was furnished with a blacksmith shop, and another small yard down town was leased for the storage of material, in order to avoid congestion in the streets.

A receiving and distribution point for the steelwork to be erected on the down-town sections was obtained by leasing a dock front and storage space at Perry Street and the North River, and, finally, a portion of the Company's surface yard at 179th Street and Third Avenue was used as a storage and distribution yard for material used on Sections Nos. 6-C and 7.

Compressors.—All riveting, drilling, etc., was done by compressed air, three stationary and thirteen portable compressors being used. The stationary compressors were Ingersoll-Rand, driven by a Nagle locomotive-type, 100-h.p. boiler; the portable compressors were Chicago Pneumatic Tool Company or Ingersoll-Rand compressors, with a capacity of 300 cu. ft. of free air per min. at a pressure of 90 lb., driven by 50-h.p. electric motors, taking current from the contact rail of the existing railways.

Travelers.—Although in some cases the erection of the steel structure was done with gin-poles and jinnywink, the greater part of the erection was done with travelers. Figs. 109 and 125 show the two general types used. When the width of the structure permitted it, the type shown by Fig. 109 was used. It spanned over two tracks, and was either supported on skids or on wheels running on a temporary track. When the working space was confined to the limits of the center track, the traveler was made narrow enough to be supported on the track stringers of the center track only, as shown by Fig. 125. The safe swing of the boom was limited in this case to the space between the outside running track, and necessitated picking up the steel between the running tracks. In order to prevent lateral swinging of the boom beyond the safe limits, which might endanger the stability of the traveler as well as the trains on the running tracks, wire ropes were attached half way up to the boom and connected to the frame of the traveler, which prevented the boom from swinging beyond certain limits. The hoists of the traveler were generally operated by electric motors, receiving the current from the contact rail of the railroad track.

Some of the steelwork was erected with a derrick-car built especially for the purpose. The derrick was built on one of the Company's standard flatcars. The A-frame was made so that it could be folded down on the derrick-car, in order to clear overhead obstructions while being transported from point to point. While being thus transported

the boom rested on another flatcar coupled in front of the derrick-car, and this flatcar was also used to transport the steel to the point of erection. The carrying capacity of the derrick-car was 8 tons, with the boom out to the side at an angle of 45 degrees.

Contractors and Cost.

Selection of Contractors.—When the work under the "Manhattan Elevated Improvements" was finally authorized to proceed, it was the desire of the Company that it should be carried out and completed speedily, safely, and without interference with the service of its trains. To secure these conditions, it was necessary to select contractors whose standing and experience would assure the success of the work, and, in order to retain their services, it was necessary to pay them for their work in an equitable manner. To advertise for bids, either on a lump-sum or unit-price basis, and then accept the lowest bidder would be a gamble, as far as the selection of the contractors is concerned, and the odds would be in favor of getting inexperienced contractors, as such men are not always able to foresee and provide against all the contingencies of the work. Even if the service of experienced contractors were secured, for instance, by accepting bids which were not the lowest, the Company would not have the freedom to select when, where, and how, the work should be carried out, so as to suit the traffic conditions on the railroads. The contractors would object to such interference, and properly so, as it would, at least in their opinion, decrease the efficiency of their work and the size of their expected profits. As an example of how the traffic conditions determined the conditions of the work, it may be mentioned that the station at 155th Street and Eighth Avenue serves the Polo Grounds, where the Baseball League games are played; the work of reconstructing this station could not commence until the baseball season was over, and had to be completed before the next season started. Another example is the carrying out of the work in Ninth Avenue; there used to be on the Ninth Avenue line a partial express service as far south as 14th Street; in order to interfere as little as possible with this service, the Company desired that the interruptions of the express service due to the reconstruction of the stations at 66th, 34th, and 14th Streets, and the grade-crossing elimination at 53d Street, should be simultaneous, and should not last more than 14 days. The work was carried out as, and within the time, described by

the Company, but not, from a contractor's point of view, in the most profitable manner. Again, when the three spans of the Harlem River Bridge were placed, it was ordered that the work should be done on a night between Saturday and Sunday after midnight, at which time the traffic was lightest; as the work depended on the tide, the placing had to wait until a Saturday night when the tide was favorable; in fact, throughout the construction, the traffic conditions were the determining factors in the progress of the work.

The desire of the Company to have the work completed speedily (the importance of which is shown by the fact that the number of passengers carried during the first year after the improvements were completed and in operation increased more than 13% over the number carried in the preceding years, during several of which the number had remained nearly constant), necessitated multiplications of the plant, tools, and materials for falsework, which contractors on a lump-sum or unit-price basis would be extremely unwilling to furnish, on account of the heavy additional cost.

Finally, the desire of the Company to carry its passengers safely during the work of construction, under contracts on a lump-sum or unit-price basis, would lead to endless controversies between the Company and the contractors, as to what constituted safety, both as to temporary supports and as to methods of conducting the work, while it was properly the intention of the Company to be the sole judge in the matter of safety.

The Company, therefore, elected to choose contractors who were known to it to possess the necessary experience to carry out the work, and to pay them the actual cost of the work plus a fixed percentage of the same to cover the use of their plant and their services. This method of payment was the only equitable one that could be devised for a construction work of this character.

The soundness of the Company's decision is shown by the facts that the work was completed in 23 months, without undue delays of the train service, and practically without injuring a passenger, although more than 600 000 000 passengers were transported on 2 000 000 trains over the lines under construction.

The work was performed under a contract, dated February 13th, 1914, with the Terry and Tench Company, Incorporated, The Snare and Triest Company, and the T. A. Gillespie Company, which last Company acted as executive. The work was distributed as follows:

All foundation work was done by the T. A. Gillespie Company; the Snare and Triest Company carried out all work, including steel erection, station finish, and track-laying on Sections Nos. 6-C and 7; the Terry and Tench Company completed all steel erection and track work on the remaining sections, and the station finish work on these sections was partly carried out by the Terry and Tench Company and partly by the T. A. Gillespie Company. The contractor's work was in executive charge of a vice-president of the T. A. Gillespie Company.

Structural Steel.—The sub-contractors for the manufacture and delivery of the steelwork, the tonnage delivered, and the prices per pound of the material delivered, are given in Table 1.

TABLE 1.—STEELWORK FOR "MANHATTAN ELEVATED IMPROVEMENTS."

Section.	Sub-contractor.	Tons.	Price per pound.
1	Milliken Brothers	5 963	0.0288
2-A	American Bridge Company	4 562	0.0293
2-B	American Bridge Company	3 820	0.0262
3	Phœnix Bridge Company	3 929	0.0239
4-A	McClintic-Marshall Company	5 544	0.0215
5-A	Pennsylvania Steel Company	1 732	0.0290
5-B	Pennsylvania Steel Company	1 440	0.0275
5-C (Structural steel)	Pennsylvania Steel Company	1 100	0.0290
5-C (Machinery)	Pennsylvania Steel Company	163	0.1045
5-D	Pennsylvania Steel Company	1 113	0.0320
6-A	L. F. Shoemaker and Company	2 926	0.0250
6-C	McClintic-Marshall Company	8 075	0.0243
7	L. F. Shoemaker and Company	1 487	0.0254
8-A	American Bridge Company	5 041	0.0245
8-B	Milliken Brothers	1 302	0.0270
8-C	Milliken Brothers	547	0.0254
10-B	Belmont Iron Works	1 200	0.0285

Rails.—The contracts for the manufacture and delivery of rails were made with the following companies:

Bethlehem Steel Company, standard rails, 2 961 520 lb., at $0.014 per lb.

Lackawanna Steel Company, standard rails, 2 916,940 lb., at $0.014 per lb.

Illinois Steel Company, manganese rails, 564 060 lb., at $0.0905 per lb.

Lumber.—The track lumber, which was all yellow pine and amounted to about 8 000 000 ft., b. m., was obtained from the D. L. Gillespie Company, of Pittsburgh, Pa., and cost $30.50 per 1 000 ft., b. m.

Other lumber, for shoring, forms, etc., was bought for $23.50 per 1 000 ft., b. m.

Material for Concrete.—The prices paid for materials for concrete delivered were as follows:

Cement	$1.70 per bbl.
Sand	1.00 per cu. yd.
Stone	1.70 " " "

Cost of Labor.—The wages paid to the men engaged on the work were at the prevailing rates, and were as follows, for an 8-hour day:

Bricklayers	$6.00
Iron workers	5.00
Iron workers apprentices	3.00 to $4.00
Carpenters	5.00
Carpenter foreman	7.00 to 8.00
Dock builders	4.00
Water-proofers	4.25
Rock drillers	3.75
Timber men	2.50
Timber men or foreman	3.00 to 3.50
Painters (structural)	2.25 " 2.50
Painters foreman	4.00 " 5.00
Blacksmiths	4.50 " 6.00
Blacksmiths' helpers	3.20
Machinists	3.50
Lead caulkers	4.50
Labor foreman	3.50 to 4.00
Handy man	2.50 " 3.00
Laborers	2.00 " 2.25
Painters for timber work	4.00
General foreman	8.00 to 10 00, straight time.
Compressor men	125.00 per month.
Hoisting engineers	30.25 to 33.00 per week.
Watchmen	12.00 " 14.00 per week.

Total Cost.—The total cost of the work done by the contractors was $10 273 636.98, distributed on the sections as follows:

Section.	Cost.
1	$1 031 678.76
2-A	773 900.88
2-B	656 784.27
3	717 651.03
4-A	692 453.22
5-A	360 804.42
5-B	325 669.73
5-C	258 458.71
5-D	187 341.30
6-A	602 671.68
6-C	1 488 380.41
7	590 966.43
8-A	854 004.07
8-B	356 153.48
8-C	108 601.83
10-B	456 919.91
General	811 196.85

The term "General" includes charges which could not be definitely distributed on the various sections. The contractors' expenses for engineering and superintendence are included in this item, and amount to $559 340.45, or about 5½% of the total cost.

Cost of Steel Erection.—The cost of steel erection varied greatly for the different sections, on account of the varying difficulties connected with the erection. Table 2 gives for each section the cost of erecting 1 ton of steel and the cost of shoring per ton of steel erected.

TABLE 2.—COST OF ERECTION.

Section No.	Cost of erection, per ton.	Cost of shoring, per ton.
1	$30.42	$12.57
2–A	21.27	5.41
2–B	13.80	1.50
3	43.71	20.18
4–A	14.22	0.88
5–A	50.85	20.61
5–B	39.62	26.53
5–C	85.76	....
5–D	32.49	6.49
6–A	28.70	17.06
6–C	39.82	10.30
7	93.94	7.34
8–A	33.41	15.33
8–B	....	....
8–C	57.80	13.90
10–B	105.24	2.68

The cost of raising the structure in Second Avenue and 125th Street, as described under Section No. 5-B, was \$6 812, and the cost of moving the tracks between 133d and 144th Streets, as described under Section No. 6-A, was \$8 474.

As an example of the cost of riveting, etc., it may be stated that, on Section No. 6-C, 365 000 rivets were driven at the cost of 14.2 cents per rivet, including overhead charges. There were four men in a gang, and each gang averaged 172 good rivets for an 8-hour day. This work was interfered with, on account of the train traffic. About 325 000 holes were drilled from the solid, mostly $\frac{15}{16}$ in. in diameter, at a cost of 15.1 cents per hole; about 40% of these holes were drilled in new steel on the street surface, the remainder in the structure. About 115 000 old rivets and bolts were cut out of the existing structure at a cost of 9.6 cents each, including replacing the rivets cut out with temporary bolts.

Cost of Placing Foundations.—The labor charges for the construction of the foundations varied considerably, according to whether the new foundations were at new locations or replaced existing foundations, and also according to the sub-surface structures encountered. On Section No. 2-A, for example, 49 foundations, each containing, as an average, 15.8 cu. yd. of concrete, were placed at new locations at \$24.50 per cu. yd., including all excavation, placing of sheathing, forms and concrete, back-filling, and repaving. Seventeen foundations under existing columns were placed at a cost of \$37.00 per cu. yd., and, in addition, the cost of shoring the structure amounted to \$210.36 for each foundation.

On Section No. 5-A, 23 foundations, each containing, as an average, 20 cu. yd., under existing columns, were placed at a cost for labor of \$18.93 per cu. yd., and the charges for shoring the structure amounted to \$255.77 for each foundation.

On Section No. 5-B, 24 foundations, each containing 7.5 cu. yd. of concrete, at new locations, were completed at a cost of \$22.47 per cu. yd., or a total of \$168.50 per pier, distributed as follows:

Superintendence:	Timekeeping .... Storekeeping .... Material checking General foreman..	...... \$21.02
	Carried fwd......................	\$21.02

Brt. fwd		$21.02
Carpenter work:	Protections	4.00
	Sheathing	20.00
	Making forms	4.00
	Placing "	7.00
	Stripping "	3.40
	Repairing "	6.00
Labor:	Excavating	50.80
	Concreting	9.90
	Back-filling	10.16
	Cleaning up	5.00
Watching		12.10
Hauling materials		15.12
		$168.50

On Section No. 1, 50 foundations, averaging 242 cu. yd. of concrete, under present columns, were placed at a cost of $22.73 per cu. yd., and the shoring labor cost $133.43 per column.

The labor charges for moving the sub-surface structures, as described under Section No. 2-A, amounted to $21 760, and, as described under Section No. 2-B, to $99 931.

Cost of Track-Laying.—The labor cost of laying center track on Section No. 6-C amounted to $1.90 per lin. ft. of straight track and $2.40 per lin. ft. of curved track.

On Section No. 2-A, 9 333 ft. of track was laid at a total cost of $20 918.28, or $2.25 per lin. ft., and on Section No. 2-B, 8 496.5 ft. of track was laid at a total cost of $20 082.98, or $2.37 per lin. ft.; these last prices include the railing for the footwalks.

Accounting.—The method of accounting for the expenditures complying with the regulations of the Public Service Commission was as follows:

Pay-rolls rendered weekly by the contractors were checked by daily force account showing the men's names, numbers, and occupations. The time was taken by timekeepers of both the contractors and the Company, who signed the sheets each day. The daily average reports also appeared on the time sheets. The time of all employees was also checked in the field by the representative of the Public Service Commission.

Material delivered on the job was entered on daily receiving sheets, being charged directly or entered into stock and charged to the job when drawn out on warehouse orders, showing the job number to which it was to be charged. The same rule as to distribution applied to the direct charge reports; all reports were signed by the engineer in charge or a duly authorized assistant. Materials taken into stock were entered on a monthly stock report furnished by the contractor and checked by the engineer.

All accounts of the contractor for pay-rolls and material received were paid for through a voucher system, sent to the engineer for checking, and the monthly estimates from which the contractor received his compensation were made up from the vouchers.

Accidents.

In addition to the usual hazards connected with erecting a steel structure, the working conditions were subjected to the dangers of frequent and fast-moving trains and of live high-voltage contact rails. A careful record was kept of all accidents that occurred during the work. The timekeepers were instructed to report immediately, on special accident report forms, all accidents that occurred, even the most trivial ones. During the whole period from March, 1914, to February 1st, 1916, when the work of the contractors was completed, there was reported 3 334 accidents, out of which 26 were fatal. As there had been in that time 1 060 000 single man working days, the average amounted to 1 accident daily per 318 men. No analysis has as yet been made of the total number of accidents, but, in April, 1915, when 1 681 accident reports had been received, which is practically one-half of the total number of reports received, an analysis was made of the accidents reported up to that time. It was found that, as an average, an accident happened daily to 1 man out of 300 employed. Of these accidents 71% did not cause any loss of time to the employee hurt and 88% (which includes the 71% mentioned) returned to the work in 2 weeks or less from the time of accident. Ten accidents were fatal.

The accidents, in accordance with their general character and their frequency, may be classified as follows:

Bruises, sprains, etc.	40	per cent.
Cuts	34	"

Accidents to the eye	12	per cent.
Accidents caused by projecting nails	6	"
Fractures	4	"
Burns	4	"

Under bruises, sprains, etc., are included all accidents resulting from having a hand or foot squeezed by materials handled, etc. Under cuts are included all open wounds, particularly those inflicted by sharp tools. Accidents to the eye include mainly small particles of foreign matter getting into the eye. The accidents under the headings, "accidents caused by projecting nails" and "fractures" are self-explanatory; the accidents under the heading "burns" were chiefly caused by short circuits from the contact rail.

As far as the causes of the accidents are concerned, they may be classified as follows:

Self-inflicted accidents	56	per cent.
Accidents caused by fellow employees	33	"
Accidents caused by trains and street cars	2	"
Accidents caused by material dropping from the structure	4	"
Accidents caused by failure of plant	3	"
Accidents caused by short circuit	2	"

The accidents included under the heading "self-inflicted" comprise all such accidents as were caused by the injured persons themselves, while under the heading "accidents caused by fellow employees" are included accidents which occurred while the injured persons were working in conjunction with other workmen. The other headings explain themselves, except the expression "failure of plant". The greater part of the accidents under this heading are such as were caused by the slipping of a ladder or the blowing out of the snap of a riveting hammer. Properly speaking, such accidents are as much failure of plant as the dropping of a boom or the breaking of a scaffold.

The following means were used to prevent accidents as far as possible. Every employee, before his services were accepted, was examined carefully by physicians, retained by the Company for the purpose, to assure that he was physically fit for the work; the foremen

were instructed to discharge any employee showing carelessness and disregard for his own or others' safety; employees under the influence of liquor were discharged and not re-employed. Flagmen employed by the Company and not by the contractors were stationed at all points where employees were on or near the running tracks; their duty was to see that all workmen were clear of the track before a train was permitted to pass. Where the structure was of unusual height, canvas or wire nets were stretched under it at the points of work. All metal tools used in the vicinity of a live contact rail were wound with insulating tape to minimize the danger of short circuits.

By far the greater number of accidents happened to the hands and feet of the employees while handling material. It would seem that such accidents could be materially reduced if only the employees could be persuaded to use proper caution. There is another class of accidents, which includes a considerable percentage of the cases, the prevention of which may be more definitely charged to the superintendence. These accidents are such as are caused by loose articles dropping from the structure, by projecting nails, and by men getting particles into their eyes when drilling steel or doing similar work. Loose articles should be secured, or picked up and placed in proper receptacles; projecting nails should be hammered down and made harmless, and drillers and others exposed to dust should wear proper protections for the eyes. It may be stated that inspections as rigid as possible along these lines were made throughout the progress of this work.

It will be noted that during the first half of the work accidents occurred at the rate of one a day for each 300 men working, but, during the latter half, they occurred at the rate of one a day for each 340 men working. Unless this in itself is an accident, it may indicate that the experience gained during the work decreased the chances for accidents occurring, and increased the knowledge as to how to prevent them.

During the work 26 fatal accidents occurred to men engaged thereon, one of which happened to one of the Company's engineers, who was struck by a train while performing his duties. Some of the accidents happened while the employee was not immediately engaged on his work, and not a single one happened when the work might be considered extra hazardous; in fact, very few accidents, fatal or other-

wise, happened when those engaged on the work realized that special precautions had to be taken. Here it may be mentioned that the placing of the three spans of the Harlem River Bridge, which work was done during the night, and once during a heavy snow storm, was completed without a single, even minor, accident, although the work, on account of the interruption it caused to traffic, was performed at the highest possible speed.

In addition to the accidents to the employees, a number happened to people passing in streets in the vicinity of the work. The greater number of these were caused by people colliding in various manners with obstructions in the streets. Other accidents were caused by material falling from the structure. The actual number of accidents to people in the streets is difficult to ascertain, as the claim that an accident had happened did not always necessarily mean that it did. There were very few and minor accidents to passengers on the platforms or in the trains.

The safety of the people in the street was provided for, as far as possible, by guarding all work with fences, by placing warning signs on all obstructions, and by stationing flagmen at all points where their services might be of value, and whenever possible by placing canvas protections under the structure, to guard against material falling to the street. The train service was protected by flagmen provided by the Company; their duty was to see that the line was clear before a train was permitted to proceed.

Organization.

The whole work, including the complete design and all the field work, was done under the direction of George H. Pegram, President, Am. Soc. C. E., Chief Engineer of the Interborough Rapid Transit Company, and was carried out in direct charge of F. W. Gardiner, M. Am. Soc. C. E., Principal Assistant Engineer, with S. Johannesson, M. Am. Soc. C. E., as General Assistant Engineer. H. C. Soest, Assoc. M. Am. Soc. C. E., was in charge of the field work of Section No. 1; H. Leser, Assoc. M. Am. Soc. C. E., of Sections Nos. 2-A, 2-B, 3, and 4-A; Mr. B. O'Rourke of Sections Nos. 5-A, 5-B, 5-C, and 5-D; Mr. J. M. Jorgensen of Sections 6-A and 6-C; Mr. O. Whitman of Sections 7, 8-C, and 10-B; and Mr. B. Helseth of Section No. 8-A.

For the contractors, Edward J. Govern, M. Am. Soc. C. E., was in general charge of the work. Holton D. Robinson, M. Am. Soc. C. E., was the contractor's Engineer, in charge of the work done by the Terry and Tench Company, and W. P. Rothrock, M. Am. Soc. C. E., of the work done by the Snare and Triest Company.

It may be stated that, throughout the work, the Engineers of the Public Service Commission, as well as the Contractors, co-operated with the Engineers of the Company to bring it to a successful finish, and that the Transportation Department of the Company, under Superintendent S. D. Smith, and the Maintenance of Way Department, under C. E. Carpenter, M. Am. Soc. C. E., gave valuable assistance in carrying out the work.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

PAPERS AND DISCUSSIONS

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ICE DIVERSION, HYDRAULIC MODELS, AND HYDRAULIC SIMILARITY

By Benjamin F. Groat, M. Am. Soc. C. E.

To be Presented January 2d, 1918.

Synopsis.

This paper treats of a new method of diverting surface water and all floating materials carried thereby for the purpose of preventing jams in canals and rivers. The paper also shows how hydraulic works may be designed by studying the performance of small-scale models.

Ice Diversion.

On many of our northern rivers, notably the Niagara and St. Lawrence, there is an immense annual crop of ice which must be taken care of in one way or another, in order to protect the various interests along the shores against damages resulting from shoves, jams, and heavy runs.

These heavy floes consist of ice in all its forms and conditions, so that no one method of treatment is likely to be entirely effective against all of them. There is, however, one property, more or less pronounced, which is common to all forms, and this fact is available toward a general solution of the problem. Ice is buoyant unless weighted down by heavier solids, such as stones frequently carried by

Note.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and, when finally closed, the papers, with discussion in full, will be published in *Transactions*.

the anchor form. Even when no heavier solids are carried, there are forms of such spongy character that the water contained by the interstices produces a water-logged variety, which, once submerged, rises to the surface very slowly.

Some years ago it occurred to the writer that the proper way to divert floating materials is to make the surface currents carry them away, rather than use a boom, which, in reality, opposes, instead of assists, the movement of floatage, and has little or no effect at all on materials suspended below the surface of the water. Following this line of thought, a patent has been applied for which covers both method and means for effecting a diversion of floating materials superficially, and a diversion of water sub-superficially, so that all, or nearly all, the water containing ice can be diverted in one direction, while the remainder can be turned into another—as into a power canal—practically free from ice and all other floating débris. In addition, it is possible to prevent jams and to control the surface currents of a river or stream so that it will be capable of carrying off all floating materials as fast as they are supplied from above, or can form within it.

As it is principally the surface currents which carry floating materials, it will be possible to measure, more or less exactly, the transporting capacity of a river at a particular place by means of the product of the width and mean surface velocity at the place. For brevity, this product may be called the "transportivity" of the river at the given place.

The transportivity of a river, then, furnishes a test of the probability of a jam or congestion of ice at any place. If it be found, for example, that the transportivity of a stream, relative to the number of square feet of floatage per second to be discharged, is great at one point and small at another a short distance below, it would appear likely that a jam might form at some intermediate place. Such a condition would exist in a reach of a river which consists of a wide deep pool fed by a broad shallow section and discharged by a narrow deep outlet of relatively large sectional area. Evidently, the pool might be supplied with floating materials more rapidly than it could discharge them, the transportivity of the feeder being much greater than that of the outlet, saying nothing of any ice which might originate in the pool itself.

The foregoing statement is not based merely on theory, but rests on firmly established facts connected with the winter conditions obtaining on our northern streams generally.

The method, then, by which a diversion of water from a river to a power canal can be made, while all the ice is carried away by the river or main stream, is to cause the river to have a high surface transporting capacity in the vicinity of the canal intake and at the same time reduce to small proportions, or even make negative, the transportivity of the canal intake itself. It is clear that no material increase in its likelihood to jam below the intake will result by reason of this arrangement.

Fig. 1 shows the writer's method and means for securing this important result. A model of an actual river has been constructed to a scale of one-hundredth the full size. This reach is marked SS', and shows the water flowing from right to left. The ice, represented by cakes of paraffin cut to scale, flows with the surface water. The intake of the canal is shown at C. In this particular instance, the entrance, or intake, of the canal, is too wide, and the first step toward reducing the transportivity of the intake is to introduce a jetty, B, extending part way across the opening. The transportivity of the intake may now be further reduced by dredging, or otherwise excavating, several channels in a transverse, or oblique, direction across the main stream and leading toward, to, or even into, the intake of the canal. As the channels must be separated by ridges, or other elevations, between them, the transportivity of the main stream will not be altered materially if the crests of the ridges are left at the original surface of the bed of the stream. This condition must maintain because the effective transverse cross-section of the main stream has not been altered materially though the effective transverse cross-section of the intake has been enlarged to any desired extent. If it should be required to increase the transportivity of the main stream, the ridges may be built to any desired elevation higher than the original bed. This joint control of the surface transporting capacities of stream and canal intake can be exercised by adjusting proportions *ad libitum*.

In many cases it will be found necessary to provide a wing, W, to prevent materials from floating into the intake around the up-stream head, and it may be of advantage to construct a jetty, D, extending from the shore opposite the intake, or to build some other kind of

structure for the purpose of narrowing the main stream in the vicinity of the intake, the object being to concentrate the surface water of the river over the diversion channels or ridges, thus reducing the length of the channels, and consequently also the excavation and ridge construction.

It may be further explained that so many transverse channels and ridges will not always be required, though the locations and dimensions of the jetties, wings, channels, ridges, or other elements for effecting a separation of the ice and canal water, must be determined in each particular instance by careful theoretical study most effectively aided by digesting the results of tests on models. As an illustration of such a change from the conditions shown in Fig. 1, the writer may refer to Fig. 2, wherein the jetty, *D,* has been removed. The illustration shows the result of a test with the same hydraulic conditions as in Fig. 1, the ice diversion being nearly as satisfactory, but the wing, *W*, requires considerable more extension into the channel of the main stream. This would be objectionable if navigation would be seriously impaired thereby.

The writer has a theory for the design, construction, and operation of such an ice and water separating and diverting works, but will not enter into many details for the present, as it would tend to distract the technical reader's attention from the main principles and facts established, and might be subject to some criticism by persons less accustomed to formal methods of thought and research. However, it may be permissible to explain briefly the operation of the sub-surface diversion channels and ridges.

It is evident that water must flow from the main stream to the divergent canal, if there is any draft by consumers along the canal. This will cause a greater or less tendency for all parts of the water in the main channel and diversion channels to flow toward the canal, as the water consumed or otherwise taken from the latter would naturally cause a fall of the water surface at the intake within the canal, and this would leave a surplus head or pressure of water at all parts in the main stream and diversion channels toward the intake.

The water in the main current, however, is flowing rapidly down the main channel because of the restricted cross-section in that direction, and the water in the transverse diversion channels is not, as it can have no component motion of any consequence at right angles

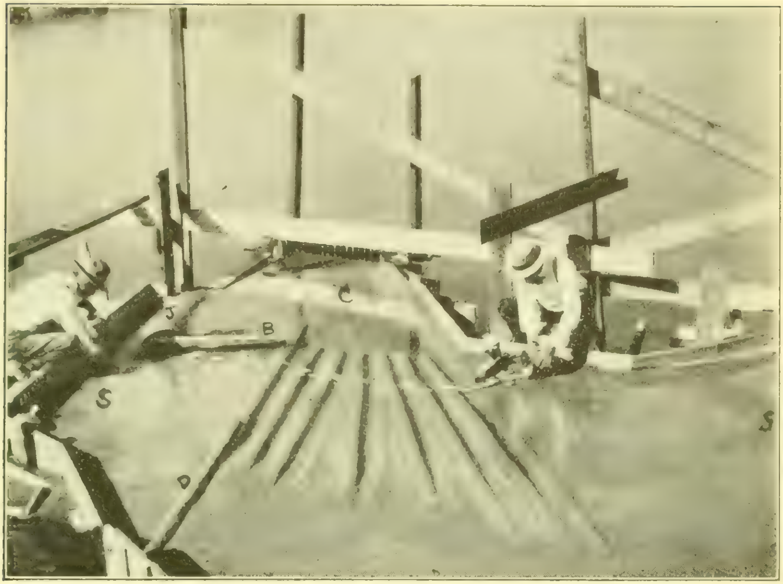

FIG. 1.—MODEL ILLUSTRATING ICE DIVERSION.

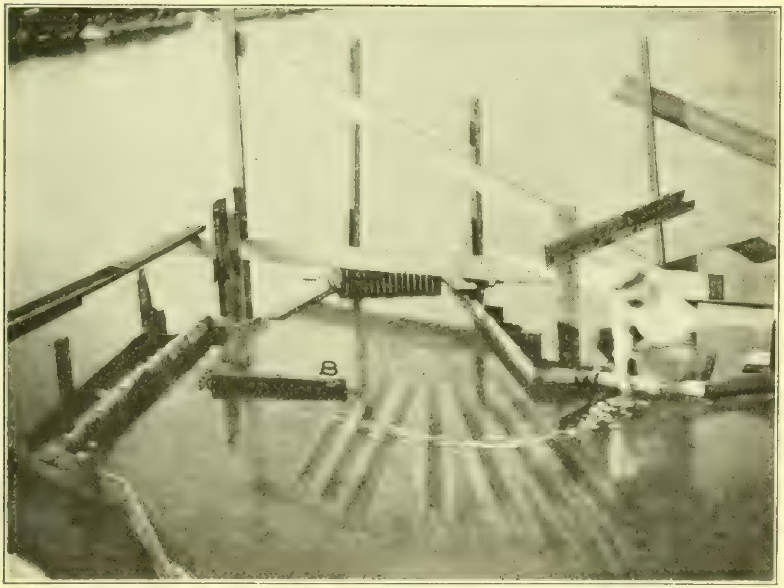

FIG. 2.

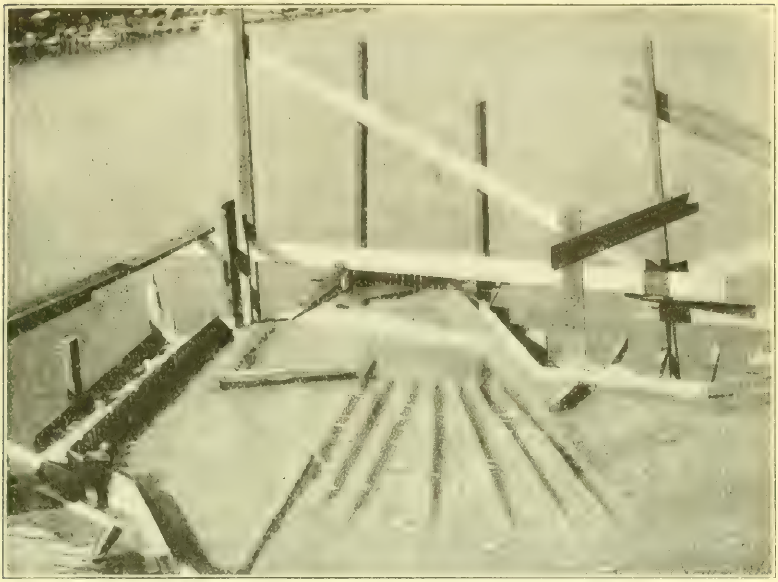

FIG. 3.

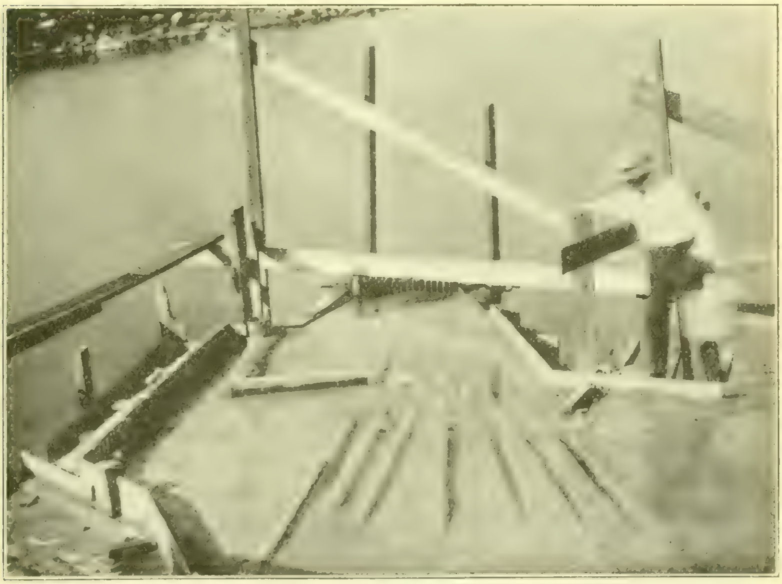

FIG. 4.

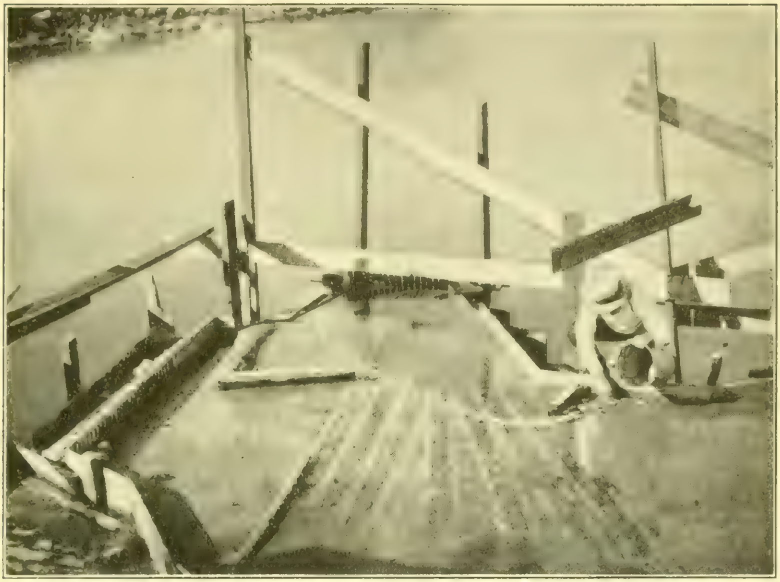

FIG. 5.

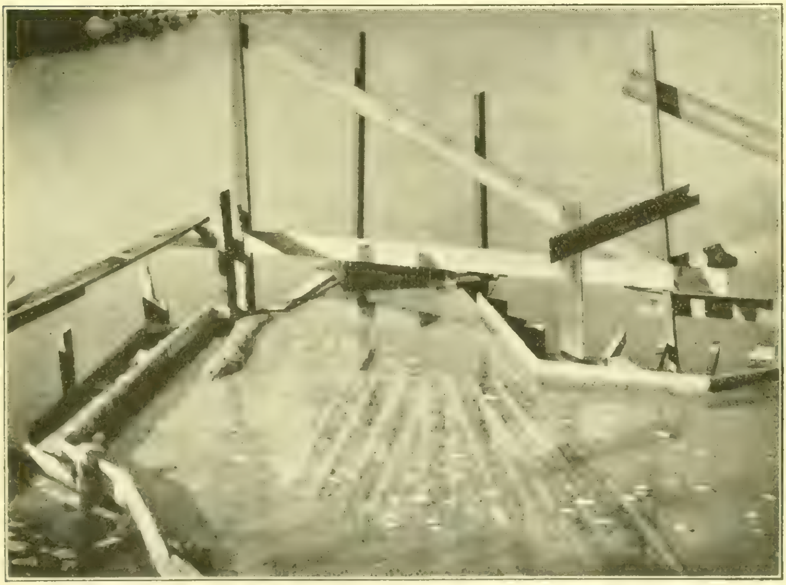

FIG. 6.

to the direction of the adjacent ridges. The water in the transverse oblique channels, between the ridges and below their crests, can flow with facility only in the direction of, or toward, the intake of the divergent canal, which it does.

Not so with the water at or near the surface of the main stream at places which are not below the crests of the ridges. This water does tend to move toward the intake, but only by reason of divergent or transverse accelerations which cause the main currents above the ridges to become more or less curved and concave toward the intake.

If the widths of the main stream and intake, the elevations of the crests of the ridges, the depths of the transverse channels, and the geographical configuration of earth and water in the vicinity of the intake have received proper attention with a view to design, alteration, construction, and operation, the surface currents of the main stream will not be impelled toward the intake sufficiently to cause any of the water at the surface to move into the intake and down the canal while it is at the surface.

It is a fact that the writer's method and invention put the forces indicated by this theory into actual operation and furnish a means for constructing and operating the intake of the canal and main channel in the vicinity of the intake, so that no surface water, and therefore no floatage, will enter the canal from the main stream. With sufficient intelligent application of the theory, the surface transportivity of the intake can actually be made negative, so that the surface of the water will flow outwardly therefrom into the main stream, the canal receiving a sufficient supply of water from the sub-surface or lower portions of the main stream to take care of the combined requirements of the canal and negative currents.

This effect can be produced with more or less intensity by making what may be called a sub-surface diversion of water to the canal from the lower portions of the main stream in greater quantity than would be necessary simply to supply the water required for the canal. The result of this is that the excess water diverted sub-superficially must cause a counter or compensating current flowing outwardly from the intake into the main stream.

Fig. 3 shows how the intake performs when ice is scattered all over the river. It may be remarked that the density of paraffin is only a trifle less than that of the heaviest ice, and about the same as

that of the lightest. When the cakes of paraffin have been used for a time they become covered with particles of dirt and grains of sand adhering to the faces, which have the effect of increasing their weight. It will be observed that no paraffin, and therefore no ice, enters the canal.

Fig. 4 shows the effect of plugging the down-stream ends of the diversion channels. Nearly all ice passing near the wing enters the canal, but it will be noticed that ice which chances to be over the unobstructed parts of the channels is not drawn into the canal but passes on down the main stream, away from the intake, beyond the influence of the canal draft. This is all clearly shown in the figure.

Fig. 5 shows the result obtained when the channels are completely obstructed by filling them with clay to the original elevation of the bed of the river. Here all the ice placed in the river at the wing passes into the canal.

Fig. 6 shows the operation of the intake in its natural condition, without diversion channels, ridges, or wings. The outlines of the diversion channels show in the illustration because the clay with which they are filled differs in color from the saw-dust concrete out of which the bed of the river was constructed. A large proportion of river ice—more than half on the average—enters the canal.

Performance of Models.

The writer has experimented with small-scale hydraulic models several times. His conclusion concerning this matter is that models perform in much the same way as the full-size prototype. In fact, there was nothing in the results of the experiments to indicate that they did not perform exactly as their prototypes. These statements apply equally to hydraulic models of all kinds, whether they be of machines, such as water-wheels and pumps; of structures, such as overflow dams, weirs and spillways; of sections of an actual river or canal; or of ships.

If, for example, a model of a section of a river has been constructed of sufficient size to prevent undue influences from properties of the fluid and materials which do not change by proper amounts with a change of scale, for example, viscosity, surface tension, etc., it will be found that the model performs almost exactly as the real section of river. Velocities, direction of flow, slopes of water surfaces, con-

figuration of eddies and bends, are all repeated in the model with great fidelity, supposing, of course, that the model has been accurately constructed and that we understand what is meant by mechanical similarity, as well as by geometrical similarity.

In the case of hydraulic models, it can be shown that homologous velocities in models of different size must be proportional to the square roots of homologous linear dimensions. When the quantities of water have been properly adjusted to comply with this requisite, it may be said that the mechanical and hydraulic conditions in the two models are mechanically and hydraulically similar, just as the configurations of fluids and solids in the models are geometrically similar.

This requirement of mechanical and hydraulic similarity has certainly been overlooked in some of the most important tests of models. Its neglect has led to the belief among many that models cannot be relied on, except as a means leading to rough approximations of doubtful value. Perhaps the tests of model water-wheels afford the most striking example. So far as the writer is aware, little heed has been taken to see that model water-wheels are tested under the proper heads; or, what is the same thing, if the head is fixed, as at Holyoke, that the size of the model wheel is properly proportioned to the head. This is saying nothing at all of the setting of the model water-wheel, which frequently bears no resemblance to the full-size setting.

Suppose, for example, that a water-wheel 110 in. in diameter is to be erected on a water fall of 180 ft. If the test is to be of a model operated under 16.5 ft. fall, then the diameter of the model should be only about 10 in., as the head for the model is only about the eleventh part of the total fall of water at the water fall. In short, the actual fall and that in the model must be in the same ratio as the linear dimensions of the homologous parts of the water-wheel and its model. It is this important requirement which is frequently lost sight of. As wheels of 30 in. or more are usually tested, it can be easily inferred that a test on such a size would result in too high a value for the efficiency in the case cited.

If the surfaces of the water passages in the model are made sufficiently smooth to represent correctly the homologous parts of the actual passages, the actual wheel and its model should perform alike. It can be shown that this simply imposes the condition that the resist-

ances to the flow of water over the homologous areas vary jointly as the homologous areas and the squares of the homologous velocities. This, we know, is nearly realized in practice. It follows that the foregoing statements are substantially correct.

The fact that homologous velocities must be proportional to the square roots of homologous linear dimensions is a fortunate matter, when the model is to be on a very small scale. Otherwise, the velocities in the model might be so small that they would not exceed Reynold's critical velocity, and thus the requirement of mechanical similarity would not be realized.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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PAPERS AND DISCUSSIONS

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THE ACTIVITIES OF THE AMERICAN SOCIETY OF CIVIL ENGINEERS DURING THE PAST TWENTY-FIVE YEARS

By Chas. Warren Hunt, M. Am. Soc. C. E.

Presented December 5th, 1917.

In 1897 a "Historical Sketch of the American Society of Civil Engineers" by the writer was published by the Society. This was issued in book form only, and a limited number sold, the proceeds being turned over to the Building Fund for the Fifty-seventh Street House. At the Washington Convention, in 1902 (the Fiftieth Anniversary of the Society), he briefly sketched the development of the intervening years. These, so far as known, form the only attempt at a connected account of the activities of the Society.

During the past quarter century many things have happened, and much has been accomplished of which there is no convenient and readily accessible record. It is true that much material, in a more or less fragmentary form, may be found scattered through the 250 monthly numbers of *Transactions* and *Proceedings* published during that period, but, even if they are all accessible in bound form, more effort and time are necessary to get at the facts than the busy engineer can afford.

In addition to this, the growth has been so rapid that only 646 (about 7½%) of the present membership of 8 544 were connected with the Society at the beginning of this period. It should be remembered also that the rate of increase in membership has been so much greater during the latter part of this period, that 5 137 (more than 65% of the increase) have joined within the last ten years.

With full recognition of the fact that statistical matter and figures are more useful in a printed than in a spoken record, it is intended to place before you this evening as briefly as possible the things which appear to be most interesting, and of which the membership in general has little if any information.

Early History.

The American Society of Civil Engineers was inaugurated at a meeting held in the office of the Croton Aqueduct Department, Rotunda Park, New York City, on Friday, November 5th, 1852. At this meeting 12 Engineers were present. Alfred W. Craven, Chief Engineer of the Croton Aqueduct, presided. The first Constitution (adopted December 1st, 1852) declared the object of the Society to be:

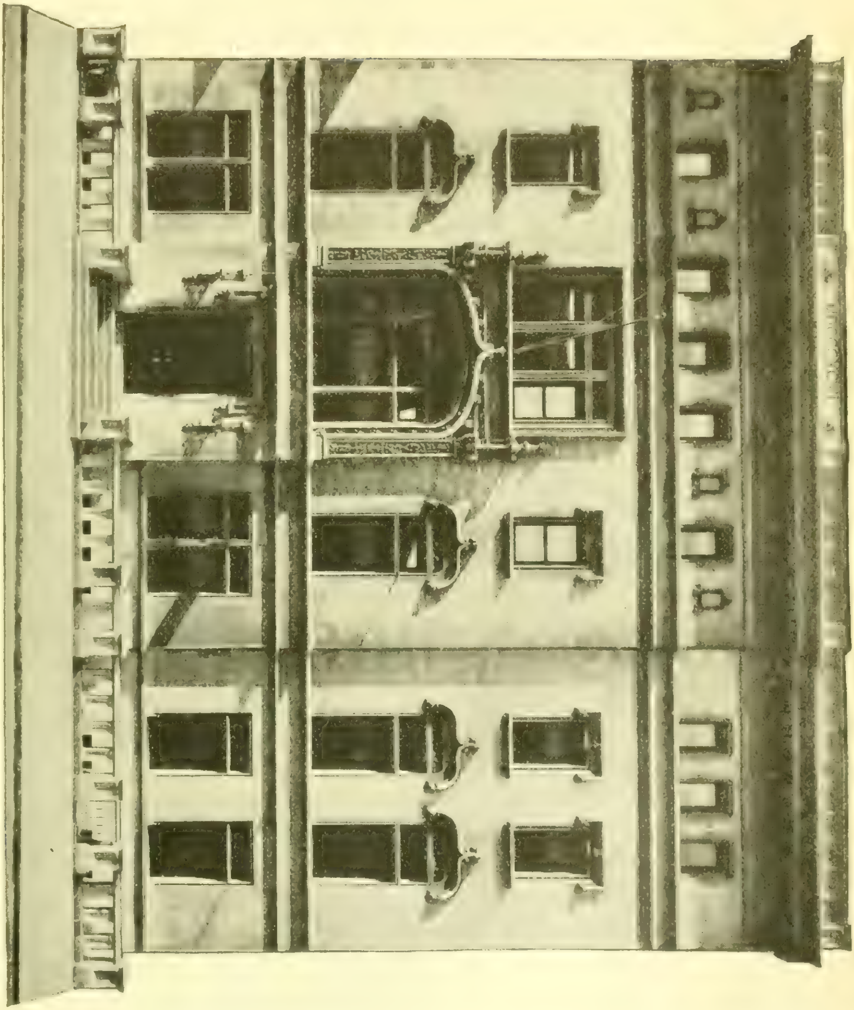
"The professional improvement of its members, the encouragement of social intercourse among men of practical science, the advancement of engineering in its several branches, and of architecture, and the establishment of a central point of reference and union for its members."

The circular issued at that time stated:

"Civil, geological, mining, and mechanical engineers, architects, and other persons who, by profession, are interested in the advancement of science, shall be eligible as members.

"It is anticipated that the union of the three branches of civil and mechanical engineering and architecture will be attended by the happiest results, not with a view to the fusion of the three professions in one; but as in our country, from necessity, a member of one profession is liable at times to be called upon to practice to a greater or less extent in the others, and as the line between them cannot be drawn with precision, it behooves each, if possible, to be grounded in the practice of the others; and the bond of union established by membership in the same Society, seeking the same end, and by the same means, will, it is hoped, do much to quiet the unworthy jealousies which have tended to diminish the usefulness of distinct societies formed heretofore by the several professions for their individual benefit."

The first professional meeting was held on January 5th, 1853. During 1853 and 1854, fourteen meetings, with an average attendance of six, were held, all in the office of the Croton Aqueduct Department. There is no record of any meeting after that of March 2d, 1855, at which the question of the securing of quarters was considered and the Society adjourned, until October 2d, 1867, when a meeting was held at the office of C. W. Copeland, 171 Broadway, New York City, at which the



House of the American Society of Civil Engineers
1897-1917

Minutes of the Meeting of March 2d, 1855, were accepted, and the object of the meeting stated to be "to take such steps as might be necessary to resuscitate the Society."

Society Headquarters.

The result of this was that the first home of the Society was in rooms in the Chamber of Commerce Building, 63 William Street, New York City, where the First Annual Meeting was held on November 6th, 1867.

In 1871 the quarters in William Street were enlarged by the renting of additional rooms, and on May 1st, 1875, new quarters were secured on the southeast corner of Broadway and Twenty-third Street.

On May 1st, 1877, the Society moved into a house, No. 104 East Twentieth Street, which it rented.

In April, 1881, a dwelling house, No. 127 East Twenty-third Street, was purchased, the first meeting being held there on May 4th, 1881, and it is of interest to note in passing that one of the Founder Societies—The American Institute of Electrical Engineers—came into being at a meeting held in that house on May 13th, 1884.

This house was occupied until 1896, when two lots, Nos. 218 and 220 West Fifty-seventh Street, with a total frontage of 50 ft., were acquired, and building operations started in December, 1896, in charge of a Building Committee consisting of George A. Just, Charles Sooysmith, Bernard R. Green, George H. Browne, William R. Hutton, Joseph M. Knap, T. C. Clarke, and Chas. Warren Hunt.

The new house was completed and formally opened on November 24th, 1897.

Owing to the growth of the Society, an additional 25-ft. lot, immediately adjoining the Society House, was purchased in 1904, and a 50% addition to the house was built. This addition was completed in the latter part of 1905, and was first used at the Annual Meeting of January 17th, 1906. The Building Committee in charge of this work consisted of Alfred Noble, S. L. F. Deyo, Nelson P. Lewis, and Chas. Warren Hunt.

The Society property then consisted of a plot of 75 ft. frontage on Fifty-seventh Street, varying in depth from about 107 ft. on the east, to about 117 ft. on the west. The House was a 4-story and basement, fireproof structure, the two lower floors covering the entire plot, and the two upper floors only the front portion. The first floor contained a

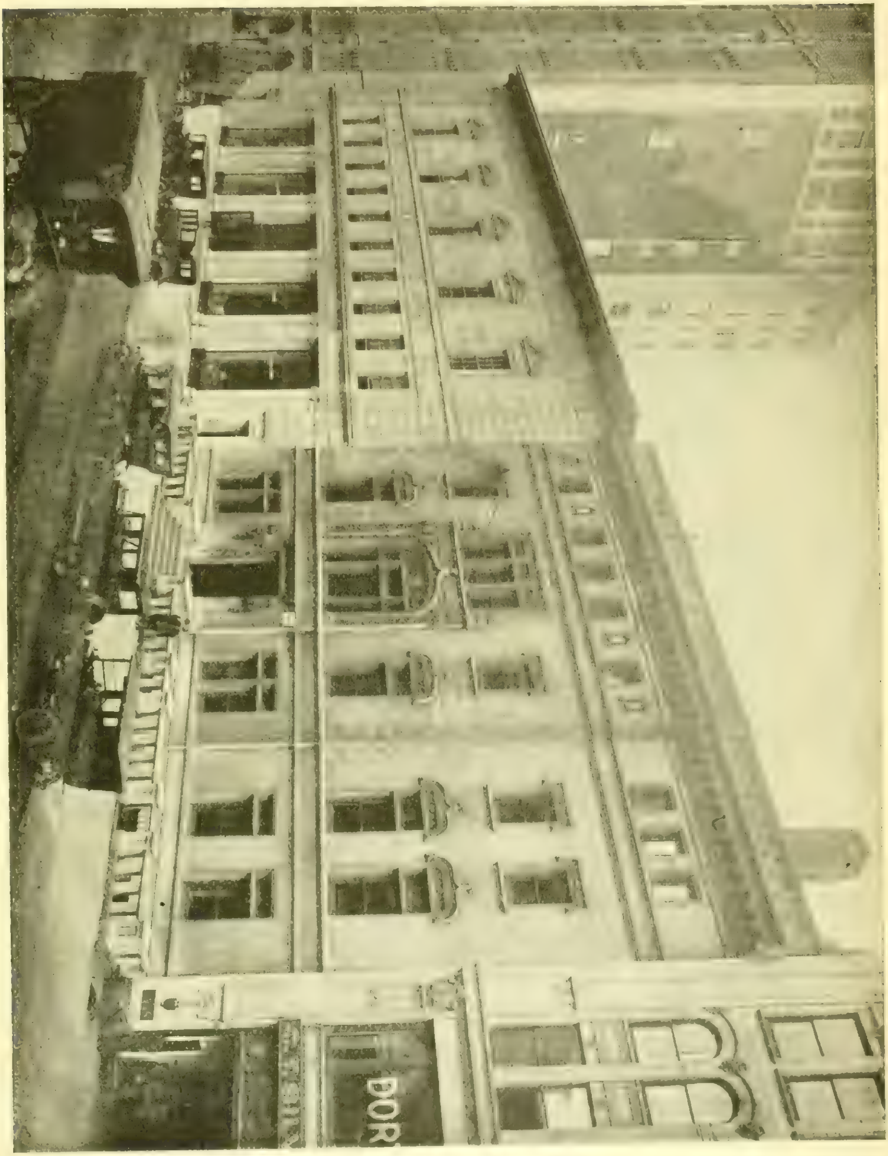
spacious foyer and three offices, one of which was used for the office of the Secretary. There was a large room in the rear called a Lounging Room, its use being principally for informal and social meetings. The main stairway gave access to the second floor on which there were in the front a large Reading Room, and in the rear an Auditorium with a seating capacity of 500. The third floor was devoted entirely to the office force, and the top floor to a double tier of book stacks with sufficient capacity for about 150 000 volumes, and with space for considerable enlargement. The building was a dignified and commodious one, and, having been specially designed for the use of the Society, proved itself adequate in every way, and, with certain additions which could have been made at any future time for the increase of space available for office and stack-room purposes, undoubtedly would have been ample for the use of the Society for many years to come. The total amount expended by the Society for the lots and building was, in round numbers, $360 000.

In February, 1903, Mr. Andrew Carnegie offered to give $1 000 000 to erect a suitable union building for the American Society of Civil Engineers, the American Society of Mechanical Engineers, the American Institute of Mining Engineers, the American Institute of Electrical Engineers, and the Engineers Club. This offer was very carefully considered by this Society, and submitted to a referendum vote of the entire Corporate Membership, the arguments for and against its acceptance being set out in an impartial manner. The result was that the membership decided, by a vote of 1 139 to 662, not to accept the offer.

The other organizations mentioned accepted. The amount donated by Mr. Carnegie was increased to $1 500 000, the result being the Engineering Societies Building, Nos. 29-33 West 39th Street, and the Engineers Club, 32 West 40th Street. The fund was divided as follows: to the three Engineering Societies, $1 050 000, to the Engineers Club, $450 000.

In 1914 the entire property of the United Engineering Society consisting of a structure of thirteen stories, built with the funds provided by Mr. Carnegie on property purchased by the three Founder Societies, had been cleared of debt.

There was, however, a strong feeling among those prominently identified with the activities of the three Founder Societies that this build-



HOUSE OF THE AMERICAN SOCIETY OF CIVIL ENGINEERS
220 WEST FIFTY-SEVENTH STREET
1897–1917

ing could not be considered a strictly representative Professional Headquarters until it housed also the oldest of the National Societies.

After several preliminary discussions of the matter by individuals, on June 9th, 1915, an informal meeting of members of all the National Engineering Societies interested in the question of co-operation of the various branches of the Profession was held, and, as a result of this meeting, the matter was taken up by the Board of Direction of this Society, and Clemens Herschel, Robert Ridgway, and Chas. Warren Hunt, were appointed a Committee to consider the question of a possible amalgamation in an Engineering Headquarters. Charles F. Loweth, Hunter McDonald, George F. Swain, and John A. Ockerson were subsequently added to this Committee, and the Board of Direction, under date of February 1st, 1916, laid the whole matter before the Corporate Membership of the Society for a referendum vote. The letter-ballot on this question was canvassed on June 15th, 1916, the result being 2 500 in favor of the acceptance of the offer of the three Founder Societies to 390 against it.

This offer, briefly stated, was as follows:

That a three-story addition be made to the Engineering Societies Building at a cost estimated at $225 000, and not to exceed $250 000. That the American Society of Civil Engineers should pay for this addition, if the cost did not exceed the latter figure, but that if that cost exceeded $250 000 the additional expense should be borne by the United Engineering Society. That the American Society of Civil Engineers would then become an equal owner in the whole enlarged property on the same terms as each of the three original Founder Societies, and would occupy as much space as it might need on two of the additional floors.

Immediately afterward the Board of Direction accepted in due form the invitation of the Founder Societies in behalf of the Society, and Clemens Herschel, J. V. Davies, and Chas. Warren Hunt, were appointed a Committee with power to carry out the agreement.

This agreement was ratified at a meeting of the United Engineering Society on August 10th, 1916. Work was begun on the necessary preliminary structural work on August 1st, 1916, under the supervision of a Building Committee consisting of one representative from each of the Founder Societies as follows: H. H. Barnes, Jr., E. Gybbon Spilsbury, Chas. F. Rand, and Chas. Warren Hunt.

Owing to the general conditions of labor and material, the cost of the addition to the building, which it was thought in 1915 was amply provided for, with all contingencies taken care of, in the estimate of $225 000, was found to be at least $50 000 in excess of the limiting figure, or $300 000. This additional cost has been borne equally by the four Founder Societies.

The total share of this Society, therefore, has been $262 500, which, together with certain additional expenses in fitting up the new quarters, cost of new furniture, and moving, will bring the total expense of our change of headquarters to approximately $280 000.

The addition, as before stated, consists of three stories. The fourteenth floor will be used as a stack-room for the United Engineering Library, headroom for a double tier of stacks having been provided. A report of the writer to the Board describes our new quarters, as follows:

"The lay-out of the floors to be occupied by this Society was made by the writer with a view to utilizing every available foot of space and to secure good light. This was the more necessary inasmuch as the floor area of these two floors is much less than that of the lower floors.

"Briefly, the Society will occupy the entire 15th floor, and about two-thirds of the 16th or top floor. In all there are eleven main rooms. On the 15th floor there are:

"(1) The office of the Secretary, entrance to which is at the right of the elevators.

"(2) The Reading Room, directly opposite the elevator, the entrance to which will be the main entrance to the Society Rooms. This room is 51 by 26 ft. and looks out over Bryant Park to the north. It is panelled in oak, and when used by our members, in connection with the Library, will, it is believed, practically take the place of the old Reading Room in Fifty-seventh Street.

"(3) The Board Room. This room, which is 43 by 24 ft. is on the south side of the building, directly opposite the Reading Room, a 6-ft. hallway separating them. This room is panelled in mahogany, and the furniture for it, which has been specially designed, is also of mahogany, and consists of 4 tables and 30 chairs. The tables are designed so that they can be placed together making a table 24 by 6 ft., or can be separated and used as units 6 by 6 ft.; and, when necessary, can be made into tables 6 by 3 ft. to set against the wall and take up very little room. In the partitions between these rooms and the hallway, two 8-ft. openings, opposite each other, with sliding doors, have been arranged, so that the two rooms can be thrown together, practically forming one large room averaging 57 by 47 ft.

ENGINEERING SOCIETIES BUILDING
33 WEST THIRTY-NINTH STREET

"(4) General Office. A large room covering the east side of the building, 59 by 37 ft. Here will be located the general office force. A service stairway, which will practically be a private stairs for this Society, gives access to the 16th floor, where, on the east side of the building, there are four small offices, one of which (5) is to be used as a Rest Room for women; (6) for the Bookkeeper; (7) Editorial Department; (8) Applications Department. Three other large rooms are available for Committee Rooms, or whatever use may develop in the future. They are (9) 24 by 20 ft., (10) 22 by 24 ft., (11) 36 by 23 ft.—these figures being approximate.

"A doorway in the hall separates that part of the 16th floor to be used by the Society from three rooms which are available for renting by the United Engineering Society, and to which access is obtained through the elevator and hallway without passing through the quarters of the Society."

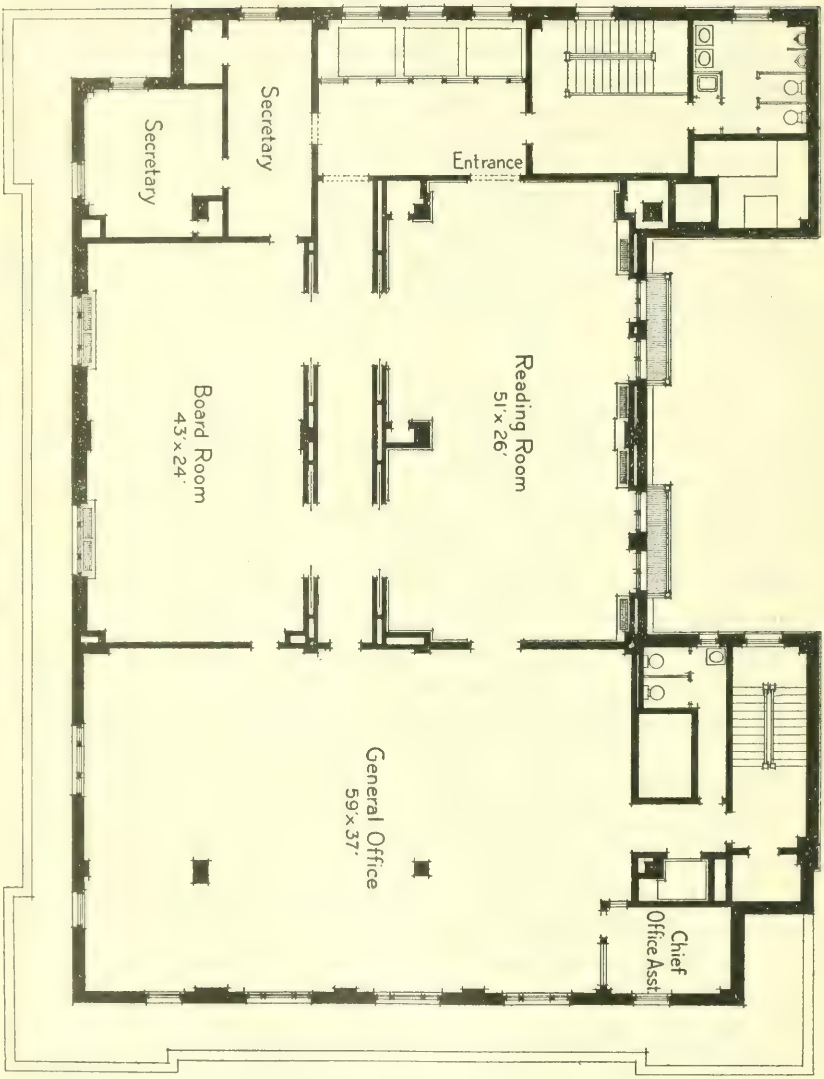
Library.

Immediately after the foundation of the Society, on January 5th, 1853, a circular was ordered to be forwarded to "All men in charge of public works, asking for printed reports, maps, plans, etc., in order to start an Engineering Library in connection with the Society." There is no record as to the response to this circular, but, naturally, very little in the way of a Library could be secured until some place was provided in which the books could be cared for, and it was not until headquarters were first established, in 1867, that the Library really had a start. After that its growth was quite rapid, when one considers that practically no books were purchased, the accessions being entirely the result of donations. Several large additions were received in the succeeding years, notably, in 1872, one from William Young Arthur, M. Am. Soc. C. E., and in 1873 one from William J. McAlpine, Past-President, Am. Soc. C. E.

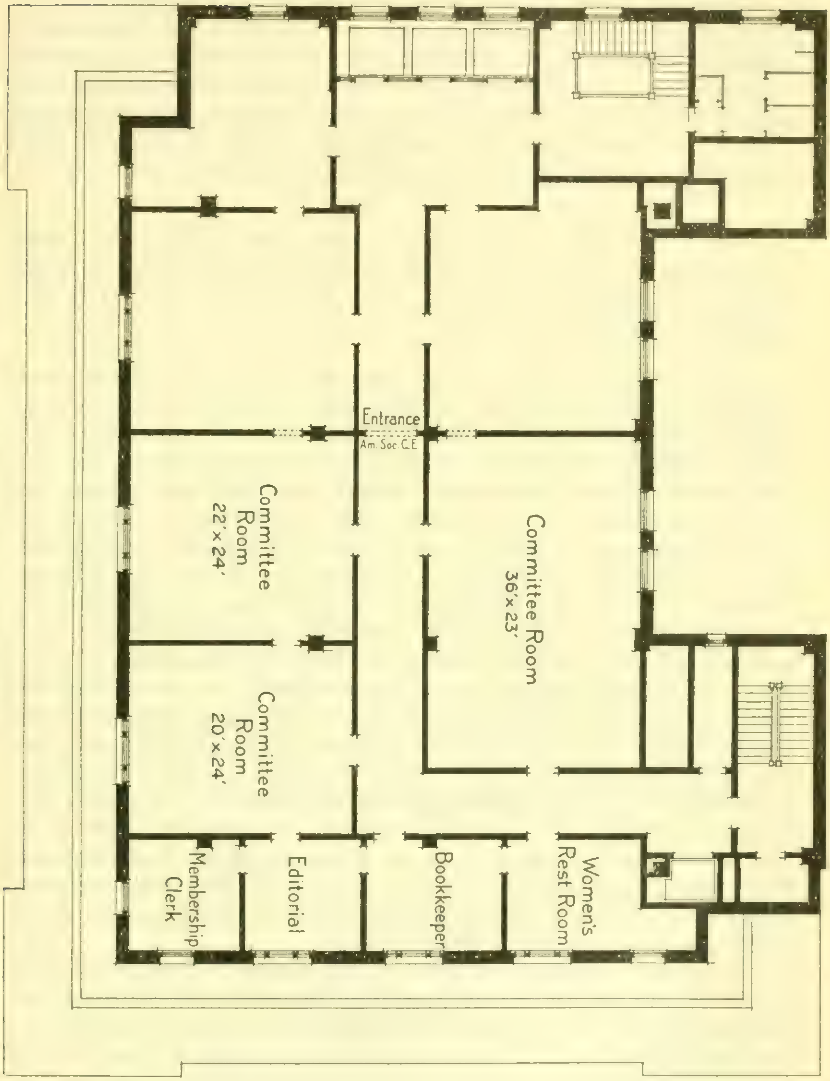
The Annual Report of the Board of Direction for 1873 gave the total contents of the Library as 3 433.

In 1873 a special committee was appointed, under the following resolution:

"Whereas, the foundation of a library and museum, which contains within itself all accessible published matter relating to the history, theory and practice of engineering, the construction and management of public improvements, and the methods and cost of manufacturing operations, with illustrations by models and samples of the results thereby obtained, must be invaluable, not only to the profession, but



American Society of Civil Engineers
Engineering Societies Building, Fifteenth Floor



American Society of Civil Engineers
Engineering Societies Building, Sixteenth Floor

to all who are interested in the pursuit or the application of practical knowledge,

"Resolved, that a Committee, consisting of the President and nine other members to be named by him, with power to fill vacancies, be appointed to devise a plan whereby such a library and museum may be founded; the funds obtained for its collection, management, increase and maintenance; a suitable place secured, where it and other possessions of the Society may be preserved and its advantages enjoyed by members and others connected therewith, irrespective of their location; * * *"

This Committee did not make a report until 1875, and it seems worth while to quote its principal recommendations, which, it is submitted, are wonderfully comprehensive, and cover the ground as thoroughly as if they had been written to-day.

"The library of the American Society of Civil Engineers should contain the literature of rational and applied science, constructive art and technology; all that has been, or may from time to time be published, relating to the history and prosecution of engineering; the maps and profiles of every canal and railroad, their complete reports, and those of municipal and state departments; descriptions of private and miscellaneous works; statistics of the material resources and development, the wealth, manufactures and commerce of countries; standard works of reference in science and art, and lack nothing published anywhere, in our own or other tongue, that in a library may aid the student or accomplished engineer seeking professional knowledge. * * * * * * Much professional knowledge recorded in the several technical journals of the day, is almost inaccessible to the busy members of a profession which allows but little time or opportunity for exhaustive reading. Complete treatises on theoretical or practical subjects, frequently published and full of matter valuable to engineers, are neither purchased or read by them. These, as issued, should form a part of the library, and its advantages be placed at the command of all connected therewith, wherever they may happen to reside, so that at their request, complete examinations on specified topics can be made, pertinent extracts copied, and proper references given.

"The plan here outlined involves the preparation of concise abstracts of new works, reports, scientific and technical journals, proceedings of societies, and other publications, as received; the whole to be classified and indexed, that a busy man may quickly learn, without the trouble and expense of looking over the vast amount of matter now published, to determine for himself, whether there has recently appeared in print anything referring to a particular subject. A serial index of current engineering and technical literature as thus described, can be comprised within a few pages issued weekly or monthly, and

would largely facilitate the dissemination of professional knowledge 'among men of practical science'.

"A skillful librarian, who knows what the library contains, and where it is to be found, can at the mere cost of the time spent, make exhaustive researches on a topic, for members, quicker and with greater thoroughness than they themselves can do it. Any one who has consulted large libraries knows that, generally, more time is spent in learning how and where to look, than in the work at hand."

In 1885, a strong effort was made to form a library for the joint use of the Civil, Mechanical, Mining, and Electrical Societies, and a committee was appointed by this Society to confer with similar committees from the other Societies; but, nearly three years later, the Chairman reported that no satisfactory progress had been made in the matter, and no further action was taken.

At the beginning of the twenty-five year period under consideration the Library had, all told, about 16 000 accessions, and five years later, when it was moved to the Fifty-seventh Street House, it contained approximately 22 000, among them being many old and rare volumes. Up to October 1st, 1916, when the Library was turned over to the United Engineering Society, the average yearly growth was 3 000, and the total number of accessions had increased to more than 89 000. More than 67 000 of these were not duplicated in the combined libraries of the Mining, Mechanical, and Electrical Societies, and these were turned over to the United Engineering Society in October, 1916. In addition, the book-stacks which had been erected in the Fifty-seventh Street House, and provided for additions to our library for many years, were donated to the United Engineering Society. They have been taken down, and are now being erected in the new "Stack Room" on the 14th floor of our new home.

The remaining 22 000 volumes have been presented to the Cleveland Association of Members. The collection is to be kept intact, and is now temporarily in the custody of the Cleveland Public Library.

In the Fifty-seventh Street House provision had been made for a commodious, up-to-date Stack Room, and, immediately upon moving in, a thorough re-classification and indexing of the Library was undertaken. The Library at that time was in an exceedingly chaotic state. No systematic index for it had ever been made, and it was a problem how it should be made efficient and available for the use of Engineers. The task fell upon the writer, and he made every effort to find out just

what had been done up to that date in the classification and cataloguing of an Engineering Library, by inquiry from available sources. A composite picture of the replies received would have read somewhat like this: "We use such and such a system, and we advise you not to." Under this condition he was thrown entirely on his own resources, and the classification which has been in use for 20 years (it is still used so far as our books, which have been transferred to the United Engineering Library, are concerned), was worked out.

In such a pioneer effort by one who, up to that time, had a very limited knowledge of Library work, it is not surprising that there were many imperfections. On the other hand, it was put together from the standpoint of an Engineer, and experience has shown that it has been a most efficient tool. This classification was used, not only to arrange books on the shelves, but also to arrange cards in the Catalogue. Many of the classes were very large, and were not sub-divided closely, and therefore the "Class Catalogue" was supplemented by a "Subject Catalogue" in which the cards were arranged alphabetically by subject. At least one card was written for every book in the Class Catalogue, and as many additional cards were placed in either the Class or Subject Catalogue as was necessary to cover its contents fully. All books were very carefully analyzed, cards being written for any sections or chapters which would be of special interest, which necessitated in some cases as many as 40 or 50 cards for one book. In addition to the two Catalogues described, there was also an "Author Catalogue" in which at least one card was filed for every book in the Library.

In 1900 the Classified Catalogue was printed and issued in a volume to all members. This book contained 700 pages, and covered about 32 000 titles. Its issue stimulated the growth of the Library to such an extent that two years later a second volume of 293 pages was issued, bringing it up to date.

During the years in which this classification was in use much experience was gained, and toward the latter part of that period an improved and extended classification was worked out by two members of the Library Staff, Miss Eleanor H. Frick, and Miss Esther Raymond, on their own initiative, and largely in their own time.* Though this classification is based on the general ideas of the writer, full credit for the work belongs to the Librarians mentioned. It is believed that the

* The two classifications are given in Appendices A and B.

publication of these two classifications will be of considerable use, not only to Technical Libraries, but to members of the Profession. As an instance of such use, it may be stated that the Committee of the Engineering Council charged with tabulating the members of the Society available for special work in connection with the War, used this classification in making up the various headings under which the members of this Society should be indexed.

In 1896, the writer in the "History" previously referred to, speaking of the Library, said:

"While it is not possible now to bring its use within the reach of members residing at a distance, it is hoped and believed that after the new house is completed arrangements can be made by which non-resident members may be able to secure data on any special points at small expense."

As soon as possible after the cataloguing had been completed, he took up the matter, and in 1902 was authorized by the Board to make searches in the Library, upon request, and to charge therefor the actual cost to the Society of the work required. About 1 000 such searches and bibliographies have been gotten out, and there is abundant evidence of the appreciation of our non-resident membership.

A number of years after this system was started, the Library of the United Engineering Society established its Service Bureau, which has been very successful; and, as our Library now forms part of the consolidation, our members will have the benefit of that service.

Local Associations.

The question of the formation of Local Associations of Members in the various centers of population was considered in a general and informal way several times prior to 1905. It was discussed at the Cleveland Convention in that year, following a report from the Secretary stating that a circular note had been forwarded to at least three Members in each of the following cities: Albany, Boston, Cleveland, Chicago, Detroit, Kansas City, Mexico, New Orleans, Philadelphia, Pittsburgh, St. Louis, St. Paul and Minneapolis, San Francisco, and Washington, setting forth the advantages of such Associations, both locally and to the Society as a whole, recommending their formation, and enclosing a draft of a proposed Constitution suitable for adoption. The Secretary reported that considerable interest had been

aroused, and that two Local Associations had been formed, one at Kansas City, Mo., and one at San Francisco, Cal.; that meetings had been held at Washington, Cleveland, Pittsburgh, Boston, St. Louis, and Philadelphia, and that a report from the three Chicago Members had also been received. The reports from Washington, Cleveland, and Pittsburgh, were non-committal. In Boston it was the unanimous opinion of those consulted that it would be very difficult to arouse sufficient enthusiasm; in St. Louis a meeting of 23 Members adopted a resolution to the effect that it was not desirable at that time to have such an organization in that city. In Philadelphia a letter-ballot was taken resulting in a vote of 42 to 14 against the proposition, and the Committee in Chicago was strongly against it.

The general idea of the organization of Local Associations of the Society, suggested by the Board of Direction, was approved by the Convention.

The writer remembers well what a hard struggle it was to overcome the many objections raised, the principal one being the fear that such Associations would injure local societies and clubs already established; but time has accomplished what then seemed impossible, and we now have Local Associations in each of the cities named except Albany, Boston, Mexico, Pittsburgh, and Kansas City. In the last named the first association was formed, but it was not successful. In addition there are 13 others, a total of 21. It is undoubtedly a fact that these Associations add strength to the Society as a whole, and are of great local benefit. Since the above was written, the writer has been informed unofficially of the formation of an Association in Pittsburgh.

An important meeting of the presidents of all the Local Associations was held at the Society House on January 19th, 1915, at which many matters of vital interest to the Society were discussed.

Membership.

Twenty-five years ago the total membership of the Society was 1 609; at the present writing it is 8 544, a net increase for that period of 6 935, the average yearly net increase having been 277. It should be noted that this increase has been in spite of the fact that the requirements have been raised during the period. The writer's opinion is that it is also due to this fact.

Finances.

As nearly as can be determined, the cash value of the property of the Society, at the beginning of the twenty-five year period under consideration, was $60 000. In a statement issued by the Board of Direction in May, 1895, when the building of the Fifty-seventh Street House was first contemplated, the available assets of the Society were given as follows:

House, 127 East 23d Street (estimate)	$60 000	
Mortgage	16 000	$44 000
Securities in safe deposit, par value		16 000
Cash, awaiting permanent investment		4 500
Amount available		$64 500

At the present time a similar statement would read about as follows:

Society House, 220 West 57th Street, cost	$360 000	
Less Mortgage	150 000	$210 000
New 39th Street Quarters, cost to the Society		267 500
Securities in safe deposit		10 000
		$487 500

The assets of the Society on the basis of this statement have increased during the past quarter century about $425 000. This, however, is very conservative, inasmuch as in the above figures the cost of the Fifty-seventh Street property is used, whereas in the statement of 1895 the value of the Twenty-third Street house was estimated, and largely in excess of the price paid for it; in addition to this, the value of the Society's one-fourth interest in the Thirty-ninth Street property is at least $250 000 more than the cost given. It would be more nearly correct, therefore, to say that the increase of property assets during this period has been $700 000.

Meetings.

During the past twenty-five years about 500 regular meetings of the Society have been held. Nearly all of these have been for the purpose of presenting and discussing professional papers and topics, and there have been 20 or 30 extra or special meetings, and about 50 meetings which are spoken of in the Constitution as for "social" purposes. There were also a number of special meetings of the Juniors of the Society.

Among the most notable events, the following might be mentioned:

The formal opening of the Fifty-seventh Street House on November 24th, 1897 was held in the afternoon. The President, Benjamin M. Harrod, of New Orleans, La., presided. The ceremonies were opened with a dedicatory prayer by the Rt. Rev. Henry C. Potter, and addresses were made by Gen. W. P. Craighill, Past-President, J. G. Schurman, LL.D., President of Cornell University, and the Hon. Joseph H. Choate.

On September 16th, 1904, a reception was given to the members of The Institution of Civil Engineers of Great Britain, who were visiting this country by invitation of the Society.

On November 30th, 1910, at the home of the Society, the John Fritz Medal was awarded to the late Alfred Noble, Past-President, Am. Soc. C. E.

On June 3d, 1912, the Society tendered a reception to the Twelfth International Navigation Congress, and on September 5th of the same year to the members of the Sixth Congress of the International Association for Testing Materials.

From 1903 to 1910 all the meetings of the John Fritz Medal Board of Award were held in the Society House, and on many occasions meetings of other societies and associations were held there by special permission of the Board of Direction.

Amendments to the Constitution.*

A revised Constitution was adopted on March 4th, 1891, the principal changes being the provision for two new grades of membership. The class of Associate Member was created, so that it would be practicable to raise the qualifications for the highest grade, and to take

* All the amendments, with a brief statement of their purport and the vote by which they were adopted or rejected, will be found in Appendix C.

care adequately of a certain class of engineers not eligible for the grade of Member, as well as to provide at the proper time a method for advancement to Corporate Membership of those in the old Junior grade who were deserving of such advancement. The requirements for the grade of Junior were lowered so as to bring them within the reach of all young men who at the beginning of their careers wished to be connected with this Society. Provision was also made for an increase in the number of Vice-Presidents and for the enlargement of the Board of Direction, so as to make it more truly representative. The respective terms of office were lengthened, and it was stipulated that members of the Board should not be eligible for immediate re-election, thus securing rotation in office.

The Report of a Committee on Revision of the Constitution, under date of November 5th, 1890, signed by W. P. Shinn, Mendes Cohen, F. Collingwood, and S. Whinery, states in part:

"It was upon the question of the duties, position and standing of the Secretary that the greatest diversity of views was found to exist. A large number of members have expressed the opinion that the Secretary of the Society, like the secretary of an ordinary business corporation, should be appointed by the Board of Direction, but those who so think forget or ignore the fact that, unlike the ordinary business corporation, the offices of President and Vice-President in this Society are of an honorary nature. The homes of these officers are most frequently in parts of the country remote from the Society's place of business, and it may often occur that they can perform but few of the executive duties. In fact the Society does not contemplate that the men whom it honors with such positions shall drop their professional duties to attend to Society work, and it certainly does not propose to pay them for doing so. The executive duties must, however, be performed by some one, and at all times. The Committee has, therefore, distinctly named the Secretary, under the President and Board of Direction, the executive officer of the Society.

"If we stop for a moment to consider the important duties to be performed by such officer, often of a delicate and confidential character, it will be seen that he should have a voice in the deliberations of the Board; for he is the source of all information, and to him must be referred the detailed investigation of every question.

"It is necessary, too, that the office should be filled by a person capable of representing the Society favorably, and deciding properly in the matters constantly arising in the intervals between the meetings of the Board; and this can only be well done by a professional man, of business experience and standing. Such a man cannot be easily

secured for any sum which the Society can at present afford to pay; nor would such a man be willing to sever himself entirely from the field of professional engagement."

Up to 1894 the office of Secretary had been filled by a general vote of the membership, but in that year an amendment was carried placing the election of the Secretary in the hands of the Board of Direction, but otherwise not changing his status. The vote on this amendment was 191 to 6. In 1895 an amendment was carried which divided the territory occupied by the Society into 7 Geographical Districts and provided for representation of each of these Districts on the Board of Direction. The vote on this ballot was 273 to 12.

The revised Constitution adopted in 1891 provided for the election of all members by a letter-ballot of all Corporate Members, 7 negative votes excluding. It also provided that the Board, upon receipt of eight requests for reconsideration of the ballot in the case of any rejected candidate, was empowered to order another ballot to be taken. On this "Reconsideration" negative ballots to the number of 10% of the votes cast were necessary for exclusion.

The small number of negative ballots necessary for exclusion on the first ballot caused trouble by the exclusion of well-qualified applicants; the reconsideration ballot also proved unsatisfactory, for the reason that the number of ballots necessary for exclusion was dependent on an unknown quantity. Under it, a candidate might be excluded with only 15 negative ballots, and another might be admitted with 40 or more negative ballots. In fact, such cases as these actually occurred.

In 1903, the number of negative ballots required for exclusion on the first ballot was increased from 7 to 20. Even this proved unsatisfactory, and in 1908 the Constitution was amended by transferring the election of members of all grades from the membership at large to the Board of Direction. The vote on this amendment was 892 to 317.

In 1915 in order to provide for a more general representation on the Board of Direction, the territory occupied by the Society was divided into 13, instead of 7, Districts, each to be represented on the Board of Direction, the vote on this question being 1 066 to 83.

A number of amendments to the Constitution have been proposed and rejected. Among the most important of these was one, submitted in March, 1907, increasing certain of the admission requirements, particularly for the grade of Member. This was lost by a vote of 429 to 847.

In 1914 an amendment was offered which would have changed the status of the Secretary of the Society by excluding him from membership on the Board of Direction. This amendment was lost by a vote of 1 343 to 1 828.

Engineering Congresses.

Three International Engineering Congresses in which the Society was active, have been held in the United States. The first was held in 1893 in connection with the World's Columbian Exposition at Chicago. This Society took charge of Division "A", Civil Engineering, the work of which was described at the joint meeting of all divisions, August 5th, 1893, as follows:

"Six sessions have been held, and the work accomplished can be best shown by the following statement: Sixty-three papers in all were presented. Of these fifty had been printed and distributed for discussion, and covered about 1 200 pages of printed matter, with numerous plates and cuts.

"The subjects treated may be classified under the following heads:

"Common Roads; Railways, Terminal Systems, Signaling, Locomotives, etc.; Cable Railways; Bridges, Substructure and Superstructure; Canals; Foundations; Surveys and Surveying Instruments; Metals—Their Treatment for Substructural Purposes; Grain Elevators; Paving Brick; Carbon—Its Use in Electrical Engineering; Electric Light Plant; Hoisting Machinery; Inland Transportation; Navigation Works; Improvement of Rivers; Improvement of Harbors; The Plant of Commercial Ports; The Laying Out of Cities; Water Works; Sewers and Sewerage; Tunnels, and The Testing of Building Material.

"Twelve countries are represented in the authorship of these papers, as follows:

Germany furnished	20	Canada	3
Mexico	6	Italy	1
Portugal	5	Australia	1
England	3	United States	18
Holland	2		—
France	2	Making a total of	63
South America	2		

"The work of translation of papers presented in foreign languages has been done in every instance by volunteers from the membership of the Society, by gentlemen thoroughly conversant with the subject under consideration.

"The interest manifested in the papers presented is evidenced by the fact that 318 engineers registered during the session of this Division, and the average attendance at each session was about 125.

"The discussions have taken a wide range, and, on account of the limited time, have been entirely confined to those presented orally. Many interesting and valuable written discussions were received, which it was entirely impossible to present at the sessions, but which will be published in connection with the papers.

"The number of valuable additions to the literature on the subjects mentioned is so great that it is impossible in this summary to do them all justice, and it is thought best not to attempt it.

"It may, however, be asserted that the results of the sessions of this Division of the Congress will be far-reaching and productive of great benefit to the profession of Civil Engineering all over the world."

The second International Engineering Congress was held in connection with the Louisiana Purchase Exposition at St. Louis, Mo., in October, 1904.

In 1903 this Society was invited by the Directors of the Louisiana Purchase Exposition to undertake the arrangements for an International Engineering Congress. Our Board of Direction appointed a Committee, and this Committee invited the co-operation of the other National Engineering Societies, but, for some reason which was never explained, they did not entertain the proposition favorably. Inasmuch as the inauguration and conduct of the proposed Congress had been placed upon this Society by the management of the Exposition, the Board determined, on January 4th, 1904, that the Society should undertake it alone, assuming the entire cost.

At that date nothing, even of a preliminary nature, had been done, and the organization, the securing, editing, and publishing of papers and discussions, as well as arrangements for meetings, devolved entirely upon the writer and his staff.

The first paper was received on March 29th, 1904, and between that date and October 1st, 1904, 83 papers were edited, printed, and circulated in advance, many discussions being received. The work of translating many of these foreign papers was undertaken by volunteers from the membership of the Society.

The Congress was held from October 3d to 8th, 1904. Its activities were divided into eight sections, 28 meetings were held, the average attendance at each being 50. In the discussion of the 38 selected sub-

jects, 97 formal papers, written by prominent specialists by invitation, were presented. In addition, 78 communications from engineers unable to be present were read, and there were 272 oral discussions at the Sectional meetings.

The proceedings were published subsequently in six extra volumes of *Transactions*, every member of the Society receiving copies of these volumes free of charge. The total edition was 4 000, and, in addition, separate pamphlets covering each of the subjects were printed, a total of 43 575 separate pieces being handled.

From foreign sources 46 out of a total of 96 papers, and 91 out of a total of 302 discussions, were furnished.

The attendance at the Congress was: from the United States 724; Canada, Cuba and Mexico 17; South America 10; Europe (13 countries) 111; Asia 10; Australia 4, a total of 876.

The total cost was $38 500, of which about $5 000 was received from subscription and sales of publications, the total net cost met by the Society being about $33 500.

The third International Engineering Congress in which the Society participated was held in connection with the Panama-Pacific Exposition, in San Francisco, Cal., September 20th-25th, 1915.

The plan of management of this Congress and the method of financing it, both of which were suggested by the writer, were as follows:

The original financial plan was that the cost should be underwritten as follows:

(1) By a general subscription from engineers residing in the Pacific Coast region........		$10 000
(2) By the five National Societies, in the following proportion:		
American Society of Civil Engineers.......	$9 000	
American Institute of Electrical Engineers..	9 000	
American Society of Mechanical Engineers..	5 000	
American Institute of Mining Engineers....	5 000	
Society of Naval Architects and Marine Engineers	2 000	$30 000
The estimated cost of the Congress was.............		$40 000

A General Committee of Management was composed of the President and Secretary of each of the four Founder Societies and of the

Society of Naval Architects and Marine Engineers, with four additional members from each Society resident in San Francisco.

The ten officers of the Societies mentioned formed a Committee on Participation, through which invitations to take part were transmitted to other Engineering organizations both at home and abroad. This Committee also arranged for providing the funds necessary to carry on the work.

The members of the Committee resident in San Francisco formed a Committee of Management to carry out the work in detail on the ground, W. F. Durand being Chairman and W. A. Cattell, Secretary-Treasurer.

This Committee took charge of the receipt, editing, printing, and distribution of the papers and discussions, which were finally issued in 13 volumes.

The total cost of the Congress was approximately $77 000. Of this amount:

Pacific Coast Engineers contributed.................	$10 413.00
American Society of Civil Engineers contributed......	7 740.00
American Institute of Mining Engineers contributed..	4 300.00
American Society of Mechanical Engineers contributed.	4 300.00
American Institute of Electrical Engineers contributed.	4 300.00
Society of Naval Architects and Marine Engineers contributed	1 720.00
Total..................................	$32 773.00

The remainder of the total expense was received from membership fees, sale of additional volumes, etc., etc.

The Annual Convention of this Society was held in San Francisco during the week before the Congress, and similar meetings of the other Founder Societies were also held, thus assuring a good attendance. This was a somewhat memorable occasion, inasmuch as a special transcontinental train for the accommodation of the members of all these organizations, and other members of the Congress, was arranged for by the Joint Committee on Entertainment and Transportation of which the writer was Secretary.

The Congress consisted of opening and closing sessions, and 51 technical meetings. The total attendance was approximately 800, and

there were about 50 official delegates. Owing to the state of war existing in Europe, the foreign participation was much more limited than had been expected when the Congress was originally undertaken.

The product of this Congress was not distributed gratis to any of the members of the Societies participating, as was the case in 1904.

Publications.

The first paper printed by the Society was an Address delivered by President James P. Kirkwood directly after the reorganization of the Society in 1867.

The number of *Transactions* for November, 1873, was the first issued. The first 57 papers, which were printed separately, make up Volume 1 and part of Volume 2. Volume 3 begins with the number of *Transactions* for May, 1874, and Volume 4 with that of April, 1875. Between that date and 1886 the number of pages published was only sufficient to fill one volume per annum, but, beginning with 1887, and continuing until 1892, two were issued yearly, the total number of volumes up to that date being 28. In 1893 two extra volumes of *Transactions* were issued containing the product of the Civil Engineering Section of the International Engineering Congress.

Up to the end of 1895 the *Proceedings* and *Transactions* were issued together in monthly numbers, and, in order to preserve them for future reference, they had to be separated and bound in individual volumes.

The difficulty with this method was that a paper intended to be submitted to the Society was not published until it had been read at a meeting, and the discussion upon it, which was limited to the few who attended the meeting or who had received advance copies, had been edited, printed, and collated. Under these conditions the membership of the Society at large never saw or heard of any paper until the discussion of it was complete, which frequently was six months, and in some cases as long as eighteen months, after the paper had been received. The result of this was that the monthly numbers of *Transactions* lacked current interest, and when received by members frequently remained in their wrappers until sent to the binder when the entire yearly volume had been received.

The writer well remembers that one of the first pieces of work assigned to him as Assistant Secretary, in March, 1892, was the getting ready for publication of the number of *Transactions* for September of the preceding year.

In 1892-95 the issue, in addition to the regular *Transactions*, of a *Bulletin* in leaflet form, calling attention to current events and giving abstracts of the papers in advance of the date at which they were to be presented, was tried. The great difficulty with this was the preparation of proper abstracts. The experience of the writer leads him to the belief that a technical abstract, in order to be really good, must be prepared by one who is expert in the particular subject treated, and that, even in this case, he must study the paper carefully and write the abstract in his own words. Any attempt to produce an abstract of a paper by quoting here and there a paragraph is not productive of satisfactory results.

In January, 1896, the publication of our present monthly *Proceedings* was begun, the technical matter contained in these being subsequently collated and published in volumes of *Transactions*.

This method was new in Society publications, and has since been adopted by others. By it the member is interested in the receipt of his monthly Number, because it contains: (1) brief accounts of Society business, including abstracts of minutes of Society Meetings both in New York and in the headquarters of Local Associations, list of additions to the membership, announcements of future meetings, and other items of general interest; (2) not only the papers to be presented, but also the discussions upon them, which are published serially until each subject is exhausted.

It is a matter of pride that, during the 22 years that this publication has been issued, it has never failed to be mailed to the membership on the fourth Wednesday of the month, although at times the issues have contained as much matter as an ordinary volume, in one case 650 pages.

In March, 1899, the writer was authorized by the Board to publish in *Proceedings* a list of current engineering articles of interest. This was started in a modest way, and was evidently found useful by the membership, because a request soon came that it be printed on one side of the page only, in order that members might cut out items which specially interested them, and use them in their own index. This list, which has been published continuously in each

monthly number of the *Proceedings* from that date, is made up from an examination of about 115 periodicals. The classification is very simple, as the list is intended to be of current interest only, and to enable an engineer to glance over each month the publications relating to his particular line of work, and to select therefrom such articles as he may read either in some convenient library or by obtaining them from the publisher.

In order to show briefly the quantity of material written, edited, and published, the total number of pages issued in the Society publications for the twenty-five years from 1867 to 1892, was 17 747 (yearly average, 710), and for the twenty-five years from that time to date has been 96 800 (yearly average, 3 872), making the total pages 114 547. The cost of the printing, binding, and postage (nearly all the postage being chargeable to publications) for the latter period has been about $724 000 (yearly average, $28 960).

The actual handling, preparation for mailing, and mailing, of all these publications has been done by the Society force during that period.

In 1911 the writer presented a Report to the Board of Direction, and subsequently to the Business Meeting of the Annual Convention of that year, suggesting that there would be many advantages if a change were made in the method of getting out our publications. The report stated that he had investigated this possibility for some time and recommended that it be tried. Briefly, the idea was to continue the publication of *Proceedings* as heretofore, but to publish only one volume of *Transactions* per annum, such volume to contain as much matter as the four that were issued at that time. This was to be accomplished by the use of thin "India", or as it is commonly called "Bible", paper. Up to 1908 two volumes of *Transactions* had been issued yearly, but, beginning with 1909, four volumes were issued per annum. (In 1910 five volumes were issued.) These volumes contained between 550 and 600 pages each. The direct benefits were fully stated in this Report.*

The recommendation was approved and the first of these thin-paper volumes was issued in 1912.

It may be set down as axiomatic in Society work that no matter what may be done, it will not please the entire membership, and this case was no exception. So many criticisms were received, with in-

* *Proceedings*, Am. Soc. C. E., Vol. XXXVII, p. 319.

quiries as to why the Society had adopted the use of "tissue" paper in its publications, etc., etc., that in April, 1914, a circular was issued asking two questions:

(a) "Shall the use of thin paper be continued in the monthly *Proceedings?*

(b) "Shall the use of thin paper be continued in the one yearly volume of *Transactions,* or shall the same number of pages be issued in *Transactions* on thick paper, in four volumes per annum"?

The result of this was that in a very large vote of about 3 000, 90% of those voting was in favor of the use of thin paper in the monthly *Proceedings,* and 95% was in favor of its use in *Transactions.*

As was foreseen, the points that appealed to the membership were the great saving to individuals in shelf room, in the cost of binding, and in economy in time by the use of one index instead of four.

Annual Conventions.

An Annual Convention has been held each year during the last twenty-five years, except in 1917, when the Convention which was to have been held in Minneapolis and St. Paul was abandoned on account of the war. Twenty-one separate localities have been visited. Two Conventions were held in Chicago, two in Niagara Falls, and two in San Francisco. All of them have been exceedingly enjoyable, have brought the members from various sections into closer contact, and have been of material benefit to individuals and to the Society.

It is perhaps worthy of notice that during this period three of these meetings have been held on the Pacific Coast, which up to 1896 was farther away from headquarters than the Society had ever held an official meeting, and that four were held on foreign soil, two in Canada, one in England, and one in Mexico.

It would extend this review too far even to touch upon the interesting events of these meetings, but perhaps it is permissible to call attention to the fact that the trip to London was made on the invitation of the Institution of Civil Engineers, that our meetings were held in the home of that Institution in London, and that the whole party had the honor and pleasure of being received by Queen Victoria at Windsor Castle. It might, perhaps, also be stated that the Mexican Convention was held by invitation of President Diaz. Members who

are interested will find quite full details of these trips in the *Proceedings.*

A special party was made up in March, 1911, to visit the Panama Canal. This was a more or less unofficial party. Two of the United Fruit Company's steamers were chartered for the occasion, one sailing from New York and the other from New Orleans, meeting at the Isthmus, and the party generally keeping together on the return. All the arrangements were made by the writer, who, unfortunately, was unable to go, due to the pressure of other duties, but he knows from what he heard from those who were fortunate enough to make it, that the trip was a specially enjoyable one.

Special Committees.

Reference should also be made to the splendid work of Special Committees appointed to investigate and report upon Engineering problems, twelve of which have made Final Reports during the period under consideration. The results of their work have been of inestimable value, but all that is possible, within the limits of this review, is to enumerate the subjects upon which such reports have been received.

Final Reports have been published on the following subjects:

Impurities in Public Water Supply; Standard Rail Sections—two Committees reported on this, one in 1893 and one in 1910—Uniform Methods for Testing Materials Used in Metallic Structures, and Requirements for These Materials to Further Improve the Grade of Such Structures; Standard Time; Regulating Practice of Engineering; Status of the Metric System in the United States; Uniform Tests of Cement; Conditions of Employment of, and Compensation of, Civil Engineers; Concrete and Reinforced Concrete; Principles and Methods for the Valuation of Railroad Property and Other Public Utilities; and Floods and Flood Prevention.

At the present time six Special Committees, all of which have presented one or more reports of progress, are investigating the following subjects:

Engineering Education; Steel Columns and Struts; Materials for Road Construction; Bearing Value of Soils for Foundations; Regulation of Water Rights; and Stresses in Railroad Track.

Medals and Prizes.

On October 1st, 1912, the Society established two additional prizes, as follows: The J. James R. Croes Medal, named in honor of the first recipient of the Norman Medal; and the James Laurie Prize, named in honor of the first President of the Society. The first consists of a medal of the value of $40, and may be awarded annually to such paper as may be judged worthy, and be next in order of merit to the paper to which the Norman Medal is awarded; the second consists of $40 in cash, with an engraved certificate signed by the President and by the Secretary of the Society. This prize also may be awarded annually, under the rules governing the award of the Thomas Fitch Rowland Prize, to such paper as may be judged worthy and be next in order of merit to the paper to which the Thomas Fitch Rowland Prize is awarded.

In a recent issue of *Engineering News-Record* the following editorial appears:

"AND THEY ARE FIGHTING IN FRANCE"

"The 'Subsidence of Muck and Peat Soils in Southern Louisiana and Florida' was the title of a paper presented two weeks ago at the meeting of the American Society of Civil Engineers. With the exception of three war addresses, equally peaceful topics have occupied the meetings since last April. The fall program, so far as announced, contains no papers bearing on the tremendous industrial and engineering problems which the winning of the war demands that we solve. This is an engineering war, yet the society seems not to recognize its opportunity."

It is unfortunate that such an improper, unfounded and sarcastic editorial insinuation should be made about an organization whose aims and objects are clearly unselfish, in a commercial publication on which the Profession in a large measure depends for its technical news.

The time for this attack upon the loyalty of this Society—just after it has become one of the Founder Societies—leaves an impression of malicious intent.

Of late all of us have heard much of the use of previously unheard of methods of warfare, and the writer feels sure that every right-minded member of our Allies of the Mining, Mechanical and Electrical Societies will unite with the members of this Society in condemnation of this misuse of editorial prerogative.

It is hoped that the following brief statement—written before the appearance of this insult to the Board of Direction and to the Membership of this Society—will be a sufficient answer.

War Activities.

As soon as war was declared, the Society placed its facilities at the disposal of the Government, and, both as an individual organization and jointly with the other Founder Societies, has done all it has been asked or permitted to do.

The value of the Engineer has been recognized to a greater extent than ever before, and in the wonderful progress made in raising, training, transporting, and maintaining the new Army of the United States, as well as in the investigation and solution of new problems, he has been a most important factor.

A Joint Committee representing the National Societies, of which William Barclay Parsons, M. Am. Soc. C. E., was Chairman, was active in securing the legislation which provided for the Engineer Reserve Corps, and Committees made up of members of these Societies have been instrumental in recruiting Engineer Regiments in many parts of the country.

In 1915, in the absence of President Marx, the writer was requested by a Sub-committee of the Naval Consulting Board to co-operate with it and representatives of other National Societies, to formulate plans for industrial preparedness. He reported to the Board of Direction, on January 17th, 1916, that a plan had been developed by the Sub-committee, acting in conjunction with the five National Societies representing the Civil, Mining, Mechanical, Electrical, and Chemical Engineers, for securing complete statistics of the industrial strength of the country. Under this plan, in each State of the Union, one representative, recommended by each of these Societies, was appointed as an Associate Member of the Naval Consulting Board, and the five Engineers thus appointed in each State constituted a Board to secure the necessary information for the Government through the aid of the more than 30 000 members of these organizations. As is well known, this great work was carried to a successful conclusion.

In these and in many other ways the Society, and its Board of Direction, has been active in the present emergency.

Every member of the Society must read with pride our "Roll of Honor", the first issue of which,* incomplete as it undoubtedly is, contains the names of 575 Engineer officers who are now serving in the Army and Navy. Since that list went to press, 148 have been added to it, and it is still incomplete. This means that more than $8\frac{1}{2}\%$ of the entire membership wears a uniform. The list, however, does not contain the names of hundreds of other members who are serving their country unobtrusively but still no less unselfishly and effectively, on Advisory Boards or simply as citizens. The writer knows of many cases where at great personal sacrifice such work has been and is now being done.

Only a few days ago a suggestion was made somewhat timidly over the telephone by J. W. DuB. Gould (one of our Members who is devoting his time to the service of the Government but who is one of those mentioned as not listed on our "Roll of Honor") that perhaps the Society might consider some arrangement by which the United States Food Administration could secure the use of the House we so recently vacated in order to carry on its work in New York City and State. The writer at once said that he believed that the Society would be glad to offer this House for the use of the Nation, for the purpose specified, free of charge.

It was not possible to get the Board together; indeed, in these busy times, a meeting of the Executive Committee is difficult to secure. By telephone, however, each available member of that Committee has given his unqualified and enthusiastic support to the proposition; the arrangement has been made, and the U. S. Food Administration Board will begin work at our old home on Friday of this week.

It is perhaps unnecessary to state that the head of this most important Board is a Member of this Society—Herbert C. Hoover.

Society Staff.

Any statement of the activities of the Society would be incomplete without special mention of the staff of the Secretary. It is not a large one. Before the transfer of the Library the total number (exclusive of Janitors and Office Boys) was 22; since that time it has been somewhat reduced. T. J. McMinn, M. Am. Soc. C. E., Assistant Secretary, and Miss Eleanor H. Frick, Chief Office Assistant, have served the Society for twenty years, and fourteen others for periods varying from

* *Proceedings*, Am. Soc. C. E., Vol. XLIII, p. 698 (November, 1917).

18 to 3 years, the average length of service of the entire force being more than 11 years. The Society owes much to the work of its employees, and the writer wishes to acknowledge publicly the faithful, industrious, efficient, and loyal service which has been rendered to the Society at all times, as well as to express his personal obligation to each of them.

FUNCTIONS OF A NATIONAL TECHNICAL SOCIETY.

The writer believes that the primary functions of a National Technical Society might be stated about as follows:

1—To advance engineering knowledge and practice.

2—To maintain the dignity and standing of the organization, and to preserve the high character and professional qualifications of its membership.

3—To keep in touch with, and to take proper action on, all matters in which the relation of the Profession to the public is involved, and to render service to the Nation when occasion demands.

4—To do whatever is possible for its Members individually, and, in general, to return to them an equivalent for the dues paid.

The latter function necessarily takes the form of providing opportunity for professional discussion, both formal and informal, which, when, as is the case in this Society, more than 80% of the membership is non-resident, must be through publications.

The use of the Library should be brought as far as possible within the reach of all, and all matters brought to the attention of the management by correspondence should be handled promptly and efficiently, including the keeping of special records of members seeking professional engagements in order that they may be placed at the disposal of inquirers for technical men in any specialty.*

Perhaps the most difficult problem is to succeed in making each member feel that he is getting as much benefit as every other member. The men who framed the Constitution of the Society were wise enough to make a decided difference in the amount of dues to be paid by Resident and Non-Resident Members, but, although the Resident Member pays 66% more than the Non-Resident, the latter is still inclined to feel that those who live near Headquarters derive disproportionate benefits, in that they may attend all meetings, use the Reading Room, con-

*While the Society has not advertised as an employment bureau, this plan has been in use for many years, and hundreds of members have been put in touch with professional opportunities.

sult the Library, and otherwise avail themselves of all local privileges.

It is not possible, of course, to arrange matters so that the Non-Resident can secure all these privileges, but, during the past twenty-five years, every effort has been made to do away with this feeling. How successful these efforts have been must be left to the individual judgment of each member, and it is hoped that what has been herein set down will aid in the formation of that judgment.

Co-operation.

Why did this Society move its Headquarters? It occupied, as has been shown, a dignified, satisfactory, commodious House, in an excellent location, which was fully paid for; its standing as an organization left nothing to be desired; its membership was increasing rapidly in all parts of the country. Why, then, give up that which had been achieved by many years of unremitting effort?

It seems to the writer that the answer is that it was the right thing to do. What if, as an organization, some sacrifices were made? What if certain details of the movement did not appeal to certain individuals? Was it, or was it not, the thing to do, from the standpoint of the Engineering Profession? The best answer to these enquiries appears to be the vote of the membership, which was 2 500 in favor of, and only 390 against the change.

Since the inception of this co-operative movement the writer has been intimately associated with it, and in close contact with the men chosen by the Founder Societies to represent the other branches of our great Profession, and can testify that the most broad-minded, earnest, and sincere spirit of co-operation has been manifest.

In a report to the Board of Direction dated September 20th, 1915, the writer said:

"The value of unity of action in all matters which affect the Profession generally must be conceded.

"For many years the undersigned has been endeavoring to bring about such a condition; he has served on the John Fritz Medal Board of Award since its organization, and as its Executive Officer for 8 or 9 years; and is now its Chairman; has, with Mr. Ridgway, represented our Society on a joint committee for the consideration of a number of subjects * * *. He has actively represented the Society on the Committee of Management of the International Engineering Congress, and has been honored by the United Engineering Society by election to, and is now serving on, the Engineering Foundation Board.

"This experience has convinced him that there should be a permanent Board or Committee, composed of an equal number of representatives of the four National Societies, to which the duty of representing the 30 000 professional men now enrolled in their membership should be given. There are many ways in which such a representative body could help the status of the engineer, in his relations with clients, employers, and the public generally, which cannot, for obvious reasons, be taken up by any one of the Professional Societies individually, and it has been his thought that an organization now exists (the United Engineering Society) which, if the representatives of the Civil Engineer are added, and its powers somewhat expanded, would be ideal for the purpose. He now believes that this matter should be the subject of discussion between the Committees of this Society and of the United Engineering Society and that the result of their deliberation should be made part of the question to be submitted to all the organizations concerned."

Two years have elapsed since this was written, and without doubt the establishment of the "Engineering Council" was intended to provide for this long felt want. Although, up to the present time, the writer has seen no reason for changing the opinion expressed—that the United Engineering Society is the organization best fitted to act on these most vital matters—it is hoped and expected that the new body will prove its value.

The years covered by this review have been indeed busy ones, not without times of serious difficulty and trial, but the bright spots after all have predominated. Association with the leaders of thought along Engineering and Scientific lines is always broadening and helpful, and the writer looks back with pleasure only on the twenty-six years devoted to the service of the American Society of Civil Engineers, during twenty-three of which he has had the honor to be its Executive Officer and a member of its Board of Direction.

APPENDIX A

CLASSIFICATION OF THE LIBRARY OF THE AMERICAN SOCIETY OF CIVIL ENGINEERS 1898–1916

BY

CHAS. WARREN HUNT
Secretary

A—RAILROADS

Aa	General
Ab	Location
Ac	Construction
Ad	Equipment
Ae	Operation
Af	Legal Documents
Ag	Reports, Company
Ah	Reports, State
Ai	Reports, Government
Aj	History

B—RAILROADS, STREET

Ba	General
Bb	Location
Bc	Construction
Bd	Equipment
Be	Operation
Bf	Legal Documents
Bg	Reports, Company
Bh	Reports, City
Bi	Reports, State
Bj	History

C—WATERWAYS

Ca	General	
Cb	Rivers	
Cc	Harbors	
Cd	Lakes	
Ce	Oceans	
Cf	Canals	
	Cfa	History
	Cfb	Location
	Cfc	Construction
	Cfd	Equipment
	Cfe	Operation
	Cff	Legal Documents
	Cfg	Reports, Company
	Cfh	Reports, State
	Cfi	Reports, Government

D—WATER SUPPLY

Da	General
Db	Water
Dc	Works
Dd	Power
De	Irrigation

E—SANITATION

Ea	General
Eb	Drainage
Ec	Sewerage
Ed	House Drainage
Ee	Sewage Disposal
Ef	Garbage Disposal
Eg	Health and Disease
Eh	Ventilation and Heating

F—BRIDGES

Fa	General
Fb	Arch
Fc	Cantilever
Fd	Draw
Fe	Girder
Ff	Lift
Fg	Suspension
Fh	Truss
Fi	Viaducts

G—MECHANICAL

Ga	General
Gb	Hydraulic Machinery
Gc	Steam Engines
Gd	Boilers
Ge	Compressed Air

H—ELECTRIC

Ha	General
Hb	Light
Hc	Power
Hd	Telegraph
He	Telephone
Hf	Various Uses

I—GAS

Ia	General
Ib	Coal
Ic	Natural
Id	Water

J—ARCHITECTURE AND BUILDING

Ja	General
Jb	Buildings
Jc	Materials
Jd	Laws
Je	Fire Prevention

K—MARINE

Ka	General
Kb	Yards
Kc	Ordnance
Kd	Naval Ships
Ke	Merchant Ships
Kf	Steam Boats

L—MILITARY

La	General
Lb	Tactics
Lc	Fortifications
Ld	Ordnance

M—MINING

Ma	General
Mb	Coal
Mc	Copper
Md	Gold and Silver
Me	Iron

N—ROADS AND PAVEMENTS
- Na General
- Nb Earth
- Nc Broken Stone
- Nd Plank
- Ne Monolithic
- Nf Brick
- Nh Stone Block
- Ni Wooden Block

O—MUNICIPAL REPORTS

P—LANDSCAPE ARCHITECTURE

Q—GEOGRAPHY
- Qa General
- Qb Physical
- Qc Statistics
- Qd Resources
- Qe Surveys
- Qf Maps and Atlases

R—SURVEYING AND DRAWING

S—SOCIETY PUBLICATIONS
- **Sa—North America:**
  - Sa1 Canada
  - Sa2 Mexico
  - Sa3 United States
- **Sb—South America:**
  - Sb1 Argentine Republic
  - Sb4 Chile
  - Sb11 Venezuela
- **Sc—Central America**
- **Sd—Europe:**
  - Sd1 Austria
  - Sd2 Belgium
  - Sd3 Denmark
  - Sd4 France
  - Sd5 Germany
  - Sd6 Great Britain
  - Sd8 Italy
  - Sd9 Netherlands
  - Sd10 Norway
  - Sd11 Portugal
  - Sd13 Russia
  - Sd14 Spain
  - Sd15 Sweden
  - Sd16 Switzerland
- **Se—Asia**
- **Sf—Africa**
- **Sg—Australia**

T—PERIODICALS
- **Ta—North America:**
  - Ta2 Mexico
  - Ta3 United States
  - Ta4 West Indies
- **Tb—South America:**
  - Tb1 Argentine Republic
  - Tb3 Brazil
  - Tb5 Colombia
  - Tb6 Ecuador
  - Tb9 Peru
  - Tb11 Venezuela
- **Td—Europe:**
  - Td1 Austria
  - Td2 Belgium
  - Td3 Denmark
  - Td4 France
  - Td5 Germany
  - Td6 Great Britain
  - Td7 Hungary
  - Td8 Italy
  - Td9 Netherlands
  - Td10 Norway
  - Td13 Russia
  - Td14 Spain
  - Td15 Sweden
- **Te—Asia**
- **Tg—Australia**

U—DICTIONARIES AND ENCYCLOPEDIAS

V—ENGINEERING HANDBOOKS

Y—GENERAL SCIENCE
- Ya General
- Yb Agriculture and Forestry
- Yc Astronomy
- Yc5 Biology
- Yc9 Botany
- Yd Chemistry
- Ye Education
- Yf Exhibitions
- Yh Geology
- Yh9 Mathematics
- Yi Metallurgy
- Yj Meteorology
- Yk Patents
- Yl Physics
- Ym Weights and Measures
- Yn Zoology

Z—MISCELLANEOUS
- Za General
- Za5 Archaeology
- Zb Biography
- Zc Charities and Corrections
- Zd Commerce
- Ze Fine Arts
- Zf Fisheries
- Zg History
- Zh Law
- Zi Manufactures
- Zj Political Economy
- Zk Religion

APPENDIX B

PROPOSED CLASSIFICATION

FOR AN

ENGINEERING LIBRARY

COMPILED BY ELEANOR H. FRICK AND ESTHER RAYMOND

OF THE LIBRARY STAFF OF THE

AMERICAN SOCIETY OF CIVIL ENGINEERS

UNDER DIRECTION OF THE SECRETARY

1916

Accompanying "The Activities of the American Society of Civil Engineers During the Past Twenty-five Years", by Chas. Warren Hunt, M. Am. Soc. C. E.

EXPLANATORY

Civil Engineering is the only class which has been expanded in detail. Certain subjects have, of necessity, been classed arbitrarily, the principal thought being utility. For instance, "Water Wheels" are placed under "Water Power" rather than under "Hydraulic Machinery"—"Locomotives" under "Railroads" rather than under "Steam Engines".

ACKNOWLEDGMENT

To acknowledge every source used in compiling this classification is not possible, because hundreds of books and indexes have been examined; mention should, however, be made of unpublished material of the Joint Committee on Classification of Technical Literature; publications of the Library of Congress, University of Illinois Extension of Dewey, and the Dewey Decimal Classifications. To John M. Goodell, and Henry S. Jacoby, Associates, Am. Soc. C. E., T. J. McMinn, and A. H. Van Cleve, Members, Am. Soc. C. E., Mr. H. E. Haferkorn, Librarian, U. S. Engineer School, Washington Barracks, and members of the Special Committee on Materials for Road Construction, of the American Society of Civil Engineers, special acknowledgment is made.

CLASSES

000 GENERAL
100 CIVIL ENGINEERING
200 MECHANICAL ENGINEERING
300 ELECTRICAL ENGINEERING
400 MINING ENGINEERING
500 METALLURGY
600 GAS ENGINEERING
700 CHEMICAL TECHNOLOGY. MANUFACTURES
800 MILITARY AND NAVAL SCIENCE
900 OTHER SUBJECTS

DIVISIONS

000 GENERAL
010 Engineering Bibliographies
020 Engineering Encyclopedias
030 Engineering Dictionaries
040 Engineering Directories
050 Engineering Societies
060 Engineering Periodicals
070 Patents
080 Engineering in General
090 Materials of Engineering

100 CIVIL ENGINEERING
110 Structural Engineering. Bridges. Buildings
120 Surveying
130 Railroads
140 Street Railroads
150 Highways
160 Hydrology. Hydraulics. Dams
170 Waterways
180 Water Power. Water-Works. Irrigation. Drainage
190 Sanitation

200 MECHANICAL ENGINEERING
210 Power Transmission. Millwork
220 Heat Engineering
230 Automobiles
240 Aeronautics
250 Hydraulic Machinery
260 Machinery for Special Purposes
270 Machine Shops
280 Miscellaneous Types of Power

300 ELECTRICAL ENGINEERING
310 Electric Measurement
320 Dynamo-Electric Machinery
330 Control
340 Transmission
350 Telephone
360 Telegraph
370 Lighting
380 Chemical Electricity. Batteries.
390 Other Uses

400 MINING ENGINEERING
410 Prospecting. Mine Surveying
420 Excavation and Working
430 Drainage and Sanitation
440 Transportation
450 Ventilation
460 Lighting. Signaling
470 Electricity in Mining
480 Accidents. Safety Measure
490 Mining Special Kinds of Ore

500 METALLURGY
510 Iron and Steel
520 Gold and Silver
530 Copper
540 Lead
550 Tin
560 Zinc
580 Other Metals
590 Assaying

600 GAS ENGINEERING
610 Natural Gas
620 Materials
630 Manufacture and Works
640 Storage
650 Distribution
660 Utilization
670 By Products
680 Management

700 CHEMICAL TECHNOLOGY. MANUFACTURES
710 Chemicals. Dyes. Paints
720 Ceramics
730 Metal Manufactures. Machinery
740 Lumbering. Wood Manufactures
750 Paper Making
760 Textiles
770 Leather Manufacture. Tanning
780 Foods and Beverages
790 Miscellaneous Industries

800 MILITARY AND NAVAL SCIENCE
810 Military Science. General
820 Fortifications
830 Ordnance
840 Naval Architecture. Shipbuilding
850 Yards
860 Navigation. Shipping
870 Naval Science. War Vessels
880 Naval Strategy and Tactics
890 Naval Organization

900 OTHER SUBJECTS
910 Philosophy.
920 Religion
930 Sociology
940 Philology
950 Natural Science
960 Useful Arts (Other than Engineering and Manufactures)
970 Fine Arts
980 Literature
990 History

SUB-DIVISIONS

To Be Used With Any Class or Sub-Class

The following nine divisions have been used as the first general sub-divisions under each main class. They may also be used with sub-divisions of any class. For instance, the sub-division *Costs and Estimates* (.04) may be applied to the general subject of Electrical Engineering (300.04), and may also be used under *Dynamo-Electric Machinery* (320), a sub-division of Electrical Engineering (320.04), and also under *Dynamotors* (322.3), which is a sub-division of *Dynamo-Electric Machinery* (322.304)—*Dynamo-Electric Machinery* being 320, sub-division *Direct-Current Machinery* being 322, and sub-division *Dynamotors* being 322.3.

.01 History
.02 Laws and Legislation
.03 Statistics
.04 Costs and Estimates
.05 Contracts and Specifications
.06 Drawings
.07 Congresses
.08 Exhibitions
.09 Tests. Laboratories

000 GENERAL
010 **Engineering Bibliographies**
020 **Engineering Encyclopedias**
030 **Engineering Dictionaries**
040 **Engineering Directories**
050 **Engineering Societies**
060 **Engineering Periodicals**
070 **Patents**
080 **Engineering in General**
081 **General Works**
085 **Ethics**
086 **Valuation of Utilities** (For Valuation of a special utility see special subject)
087 **Industrial Management**
.1 Organization
.2 Efficiency Engineering
.21 Scientific Management. Motion Study
088 **Construction Work. Contracting**
.1 Contracts and Specifications. General Works (For Special Contracts and Specifications, *See* .05, under that subject)
.2 Organization
.3 Methods
.31 Timekeeping,etc.
.4 Inspection
.5 Contractors' Plant
089 **Excavation. Earthwork** (*See also* 420, Excavation and Working, under Mining Engineering)
.1 Earth Excavation
.2 Rock Excavation
.3 Excavating Machinery
.31 Steam Shovels
.32 Ditching and Trenching Machinery
090 **Materials of Engineering** (*See also* 111, Mechanics of Materials)
091 **Engineering and Testing Laboratories**
.1 Laboratory Manuals
.2 Testing Machines and Appliances
.3 Methods of Testing

091 **Engineering and Testing Laboratories** *(Continued)*
.31 Selection of Test Pieces. Influence of Temperatures, etc.
.32 Weathering
.33 Elastic Limit Tests
.34 Tension, Compression, Torsion, Flexure, Shearing
.341 Tensile Tests
.342 Compression Tests
.343 Torsion Tests
.344 Flexure Tests
.345 Shearing Tests
.346 Repeated Stress Tests
.35 Impact. Repeated Shock Tests
.36 Hardness Tests
.37 Special Tests (Varying for different materials)
.38 Tests on Special Shapes and Forms
.39 Other Tests

092 **Timber. Strength and Testing**
.1 Influence of Temperature
.2 Weathering. Decay and Preservation
.3 Elastic Limit Tests
.4 Tension, Compression, Torsion, Flexure, Shearing
.5 Impact. Repeated Shock Tests
.6 Hardness Tests
.7 Special Tests for Timber
.8 Special Shapes
.81 Posts
.82 Columns
.83 Shafts
.84 Cylinders. Pipe, etc.
.9 Descriptions of Various Kinds of Timber (Arranged Alphabetically)

093 **Masonry Materials**
.1 Stone
.11 Influence of Temperature
.12 Weathering
.14 Tension, Compression, Shearing, Crushing
.15 Impact
.16 Hardness Tests
.17 Special Tests for Stone, etc.
.18 Special Shapes and Forms
.19 Descriptions of Kinds of Stone (Arranged Alphabetically)
.2 Brick
.21 Influence of Temperature
.22 Weathering
.24 Crushing Tests
.27 Special Tests
.271 Rattler Tests
.272 Absorption Tests
.3 Tile
.4 Terra Cotta
.5 Lime. Mortar
.6 Cement
.61 Influence of Temperature. Selection of Test Pieces
.62 Weathering
.63 Tension, Compression, etc.
.65 Impact Tests
.66 Soundness. Constancy and Time of Setting
.67 Special Tests
.671 Fineness of Grinding
.672 Accelerated Tests

094 **Concrete. Strength and Testing**
.1 Influence of Temperature and Mixing. Selection of Pieces
.11 Concrete Aggregates
.111 Sand
.112 Gravel
.113 Slag
.114 Water
.12 Forms. Removal of Forms
.13 Effects of Freezing on Concrete
.2 Weathering. Concrete Finishing
.21 Action of Salt Water
.22 Action of Gases and Other Chemicals
.23 Water-proofing
.24 Concrete Finishes. Stucco
.3 Elastic Limit Tests
.4 Tension, Compression, Torsion, Flexure, Shearing
.5 Impact
.6 Hardness

112	**Theory of Design. Stresses and Strains** (*Continued*)	
.23		Impact Loads
.24		Wind Pressure
.25		Snow Loads
.3	Iron and Steel Construction (General. For an Iron Truss Bridge, (*See* 112.92)	
.4	Masonry Construction (General)	
.41		Stone Construction (General)
.42		Brick Construction (General)
.5	Concrete Construction (General)	
.6	Reinforced Concrete Construction (General)	
.7	Timber Construction (General)	
.8	Esthetics of Design	
.9	Design of Special Types	
.91		Arches. Arch Bridges
.911		Metallic
.912		Masonry
.913		Concrete
.914		Reinforced Concrete
.92		Trusses. Truss Bridges
.921		Simple
.922		Continuous
.93		Girders. Girder Bridges
.94		Movable Bridges
.941		Swing
.942		Lift. Bascule
		Turntables (*See* 133.83)
.95		Cantilever Bridges
.96		Suspension Bridges
.97		Viaducts
.98		Other Forms of Bridges
.981		Ferry Bridges. Transporter Bridges
.982		Pontoon
.983		Trestles
.984		Tubular
.99		Roofs. Domes
113	**Foundations. Bridge Piers and Abutments**	
	General	
.001		Exploration of Site
.0011		Borings
.0012		Supporting Power of Soils
.002		Theory of Design
.0021		Loads
.0022		Area of Foundation Required
.0023		Center of Pressure and Center of Base
.1	Underpinning. Shoring	
.2	Spread Foundations	
.21		Masonry Footings
.211		Stone
.212		Brick
.213		Concrete
.214		Reinforced Concrete
.22		Timber Grillage
.23		Steel Beam Grillage
.24		Eccentric Footing
.25		Cantilever Footing
.26		Inverted Arch
.3	Piles and Pile-Driving	
.31		Timber Piles
.311		Mechanical Protection (Cement Mortar, Galvanized Iron, Pipe, etc.)
		Preservation of Timber (*See* 092.2)
.32		Concrete Piles
.321		Pre-moulded
.322		Cast in Place
.33		Metal Piles
.331		Disk Piles
.332		Screw Piles
		Sand Piles (*See* 113.913)
		Pneumatic Piles (*See* 113.6)
.34		Sheet-Piles
.35		Pile-Drivers
.36		Driving by Water-Jet
.37		Bearing Power of Piles
.4	Coffer-Dams	
.41		Earth
.42		Sheet-Pile

113 **Foundations. Bridge Piers and Abutments** (*Continued*)
.43 Crib
.44 Movable
.45 Miscellaneous
.5 Box and Open Caissons
.51 Box Caissons
.52 Single-Wall Open Caissons
.53 Cylinder Caissons (Open Wall. Wood. Iron. Masonry)
.54 Open Caissons with Dredging Wells
.55 Hydraulic Caissons
.6 Pneumatic Caissons
.61 For Bridges
.62 For Buildings
.63 Caisson Disease
.7 Pier Foundations in Open Wells
.8 Bridge Piers. Bridge Abutments
.81 Width of Waterway
.9 Miscellaneous Processes
.91 Consolidating the Soil
.911 By Adding Sand
.912 By Driving Wooden Piles
.913 By Using Sand Piles
.914 By the Compressol System
.92 Drainage
.93 Freezing
.94 Grouting
114 **Retaining Walls**
115 **Bridges in General** (For Design, *See* 112)
.1 For Special Uses (For general material only. Class with special type if possible)
.11 Highway Bridges
.12 Railroad Bridges
.13 Military Bridges (*See also* 112.982 and 116)
.14 Park and Ornamental Bridges
.2 By Special Location
.21 Skew Bridges
.3 Of Special Materials (Divided like 090. Class with special type if possible)
.4 Location. Approaches
.5 Selection and Comparison of Types
Foundations. Bridge Piers and Abutments (*See* 113)
116 **Bridge Construction** (Put here only practical material on Erection, Maintenance, Failures, etc., and Descriptions of Bridges actually Built)
.001 Shop Practice. Shop Inspection
.002 Layout of Plant
.003 Erection. Falsework. Erecting Equipment
.004 Bridge Floors
.005 Bridge Painting
.008 Other Details
.009 Maintenance (Inspection, Lighting, etc.)
.1 Arch Bridges
.2 Truss Bridges
.3 Girder Bridges
.4 Movable Bridges
.5 Cantilever Bridges
.6 Suspension Bridges
.61 Towers
.62 Anchorage
.63 Cables
.64 Stiffening Trusses
.7 Viaducts
.9 Other Forms
.91 Ferry Bridges. Transporter Bridges
.92 Pontoons
.93 Trestles
.94 Tubular
117 **Building Construction. Buildings**
.003 Erection of Steel Buildings
.02 Building Laws
.1 Details of Construction
.11 Floors
.12 Ceilings
.13 Stairs
.14 Windows and Doors
.15 Chimneys and Flues

117 **Building Construction. Buildings** (*Continued*)
.16 Roofs and Roofing (For Design of Roofs, *See* 112.99, Roofs, under Theory of Design)
.17 Vaults
.18 Walls. Partitions
.19 Other Details
.191 Fire Escapes
.192 Scaffolding
.193 Elevator Shafts
Ventilation and Heating (*See* 227)
.2 Educational, Government and Scientific Buildings. Exhibitions, etc.
.3 Business and Commercial (*See also* 117.7, Factories and Mill Buildings)
.31 Office Buildings
.32 Stores, etc.
.33 Markets
.4 Domestic Architecture. Residences
.41 Houses. Bungalows
.42 Tenement Houses. Housing of the Poor
.421 Model Factory Towns
.43 Apartment Houses
.44 Hotels
.5 Storage Buildings
.51 Bins (*See also* 133.86, Sand Plants and Bins, under Railroads)
.52 Coal Storage Plants
.53 Grain Elevators
.54 Ice Houses
.55 Lumber Sheds
.56 Warehouses
.6 Hospitals and Asylums
.7 Factories and Mill Buildings
.9 Other Buildings
.91 Prisons. Reformatories (*See also* Prisons, under Social Sciences)
.92 Churches. Monuments. Mausoleums
.93 Recreation Buildings
.931 Theatres
.94 Baths
.95 Public Comfort Stations
.96 Wash Houses. Public Laundries

118 **Fire Prevention and Protection**
.1 Fire Resistive Construction
.2 Fire Prevention and Extinction
.21 Fire Alarms
.22 Fire Extinction
.221 Automatic Sprinklers
.222 Chemical Engines. Chemical Systems
.223 Fire Engines, etc.
.224 Fire Boats
.3 Conflagrations (Arranged Alphabetically by Place)
.4 Fire Departments

119 **Other Structures**
.1 Stadium (*e. g.*, Yale Bowl)
.2 Subways for Passenger and Vehicular Traffic
.3 Pipe Subways

120 **Surveying. General**

121 **Instruments**
.1 Tapes, Chains, Rods, etc.
.2 Compass
.3 Transit and Theodolite
.31 Solar Attachment
.4 Sextant
.5 Photo Theodolite
.6 Plane Table
.7 Level
.71 Aneroid Barometer
.8 Drawing Instruments
.9 Other Instruments

122 **Land Surveying**

123 **City Surveying**

124 **Topographic Surveying**
.9 Photographic Surveying

125 **Geodetic Surveying**
.1 Triangulation. Base Lines
.2 Stations, Towers, etc.
.3 Adjustments of Errors
.4 Astronomical Observations

126 **Leveling**

127 **Plotting**
.1 Projections
.2 Lettering
.3 Photographic Reproductions. Blue Prints
130 **Railroads**
General
.01 History of Railroad Engineering
.011 History of Individual Railroads (Arranged by Railroad)
.02 Laws and Legislation. Legal Documents
.021 Of Individual Railroads (Arranged by Railroad)
.022 Of States or Provinces (Arranged by Name)
.023 Of Governments (Arranged by Name)
.03 Statistics
.04 Costs and Estimates
.05 Contracts and Specifications
.06 Drawings
.07 Congresses
.08 Exhibitions
.09 Tests. Laboratories
.9 General Descriptions of Railroads (Arranged by Name and Place)
131 **Location**
.1 Preliminary Plans. Economics of Location
.11 Land Grants
.2 Surveying
.21 Curves. Transition Spirals
132 **Construction of Roadbed** (To sub-grade)
Earthwork (*See* .089)
.1 Drainage. Culverts
.2 Tunnels and Tunneling
.21 Masonry Lining
.22 Grouting
.3 Grade Crossings. Track Elevation and Depression
133 **Construction and Maintenance of Permanent Way**
.1 Roadmasters' and Trackmasters' Regulations and Manuals
.2 Gauge
.3 Ballast
.4 Ties and Sleepers
.5 Rails
.51 Rail-fastenings
.6 Turnouts. Frogs and Switches. Track Crossings
.7 Removal of Wrecks, Snow, and Weeds
.71 Removal of Wrecks
.72 Removal of Snow
.73 Removal of Weeds
.8 Track Accessories
.81 Track Tanks. Fuel and Water Stations
.82 Ash-pits, etc.
.83 Turntables. Transfer Tables
.84 Fences, Cattle-guards, Snow Guards
.85 Snow Sheds
.86 Sand Plants and Bins
.9 Yards and Terminals
134 **Buildings**
.1 Passenger Stations
.2 Freight Stations
.3 Engine Houses. Roundhouses
.4 Shops
.5 Coal Sheds
135 **Rolling Stock**
.1 Cars
.11 Car Details
.111 Wheels
.112 Axles
.113 Bearings and Lubrication
.114 Springs
.115 Couplings
.116 Brakes
.117 Furnishings
.118 Sanitation, Heating, Lighting, Ventilation
.12 Passenger Cars
.13 Baggage and Mail Cars
.14 Freight Cars
.15 Work Cars
Motor Cars (*See* 136.6)
.2 Locomotives (Put here Manuals for Locomotive Engineers and Firemen. *See also* Electric Locomotives, 136.5)

135 **Rolling Stock** (*Continued*)
.21 Design and Construction
.22 Details (Frames, Boilers, etc. Divide if necessary)
.23 Types
.24 Locomotive Tenders
.25 Locomotive Shops and Works
.26 Locomotive Maintenance and Repair. Inspection
.27 Locomotive Performance
136 **Use of Electricity. General Works on Electric Railroads**
.1 Power Requirements. Sub-Stations (For Power Plants, *See* 321, Electric Plants, under Electrical Engineering)
.2 Power Transmission
.21 Overhead Trolley Systems
.22 Third-Rail Systems
.23 Other Systems
.3 Track } Material to be classed here only when differing in
.4 Cars } construction from ordinary track and cars
.5 Electric Locomotives
.6 Railway Motor Cars
.7 Rolling Stock Accessories and Parts
.71 Motors, Controllers, etc.
.8 Locomotive and Car Wiring
137 **Operation**
.1 Signals and Signaling
.11 Hand Signals
.12 Telegraph and Telephone Systems
.13 Block Systems
.14 Interlocking Systems
.2 Train Movement
.21 Train Resistance
.22 Train Speed
.23 Train Load
.24 Train Running
.241 Travelers' Guides. Time-Tables
.3 Railroad Accidents. Safety Measures
.4 Traffic
.41 Station Management
.42 Passenger Traffic
.43 Baggage
.44 Express
.45 Mail Service
.46 Freight
.5 Rates
.51 Passenger Tariffs
.52 Freight Tariffs
.6 Finance
.61 Capitalization
.62 Valuation
.63 Accounting
.64 Receivership and Reorganization
.65 Clearing House
.7 Organization of Staff and Force
.71 Service Rules and Regulations
.72 Wages
.73 Strikes
.8 Federal and State Relations
138 **Reports**
.1 Company Reports
.2 State Commission Reports (Enter here only serials or reports so general in nature that they cannot be classed elsewhere)
.3 Government Reports (Above note applies also to this class)
139 **Miscellaneous Kinds of Railways**
.1 Mountain
.11 Cable
.12 Rack
.2 Monorail
.3 Aerial Tramways
.4 Industrial Railways
.5 Ship Railways
140 **Street Railways. Elevated Railways. Subways**
General
.01 History
.02 Laws and Legislation
.03 Statistics
.04 Costs and Estimates

140 **Street Railways. Elevated Railways. Subways** (*Continued*)
.05 Contracts and Specifications
.06 Drawings
.07 Congresses
.08 Exhibitions
.09 Tests. Laboratories
.9 General Descriptions of Street Railways (Arranged by Name and Place)
141 **Location**
.1 Preliminary Plans. Promotion
.2 Surveys
142 **Construction in General**
.1 Surface Railways
.2 Elevated Railways
.3 Subways
.4 Freight Subways
143 **Construction of Track. Maintenance**
.1 Paving Between Tracks
.2 Rails. Bonding
.3 Electrolysis. Leakage
144 **Structures and Buildings**
.1 Passenger Stations
.2 Car Barns
.3 Shops
.9 Other Structures
145 **Rolling Stock**
.1 Cars
.2 Car Heating and Lighting
.3 Car Painting
.4 Car Maintenance
.5 Accessories and Parts. Street Railway Motors, etc.
146 **Traction**
.1 Electric Power Requirements
Electric Power Plants (*See* 321)
.2 Electric Power Transmission Systems
.21 Overhead Trolley Systems
.22 Underground Conductors (Conduits)
.23 Third Rail Systems
.24 Other Systems
.3 Traction Other Than Electric
.31 Cable
.32 Compressed Air
.33 Horse
147 **Operation**
.1 Signals and Signaling
.2 Movement of Trains and Cars
.3 Accidents
.4 Traffic
.5 Finance and Accounts
.51 Capitalization
.52 Valuation
.53 Accounting
.6 Organization and Employees
.7 Municipal and State Relations
148 **Reports**
.1 Company
.2 City
.3 State
.4 Government
150 **Highways** (General Works and Treatises classed here)
General
.01 History
.011 Descriptive Material (Put here material on roads of "New Jersey", "Lincoln Highway", etc., arranged by locality)
.02 Legislation
.021 Municipal
.022 County
.023 State
.024 National
.03 Statistics
.031 Local
.032 County
.033 State
.034 National
.035 Automobile and Motor Truck
.04 Costs and Estimates

150 **Highways** (*Continued*)
.05 Contracts and Specifications
.06 Drawings
.07 Congresses
.08 Exhibitions
.09 Tests. Laboratories
151 **Administration. Reports**
.1 Organization
.11 Civil Service Registrations
.2 Maintenance Systems
.3 Labor. Convict Labor
.4 Inspection
.5 Municipal Reports
.6 County Reports
.7 State Reports
.8 Government Reports
152 **Economics**
.1 Financing
.11 Administration and Engineering
.12 Construction of Highways
.13 Maintenance of Highways
.14 Labor Tax
.15 Bonds
.16 Assessments
.17 Appropriations
.2 Accounting
.3 Comparison of Different Roads and Pavements
.4 Effect of Road Improvements on Social and Business Conditions
.5 Traffic
.51 Census
.52 Tractive Resistances
.53 Costs of Transportation
.6 Contracting
153 **Design**
.1 Preliminary Investigations
.2 Highway Systems
.21 Municipal
.22 County
.23 State
.24 Government
.3 Surveys
.4 Mapping
.5 Location and Grades
.6 Widths
.7 Crowns, Cross-Sections and Street Intersections
.8 Bridges and Fords (Location only. For Design and Construction of Bridges, *See* 110)
.9 Miscellaneous
154 **Highway Details**
.1 Grade Crossings, Elimination of (*See also* 119.2)
.2 Subsurface Structures
.21 Pavement Openings
.3 Sidewalks
.31 Gravel
.32 Cement-Concrete
.33 Tar Concrete
.34 Asphalt
.35 Brick
.36 Stone Flags
.37 Stone Blocks
.38 Wood
.39 Other Materials
.4 Curbs
.41 Stone
.42 Cement-Concrete
.5 Fences, Guard Rails and Walls
.6 Road and Street Signs
.7 Car Tracks, Paving (*See also* 143.1)
.8 Trackways
.9 Miscellaneous
Roadsides, Esthetic Treatment (*See* 971, Landscape Architecture)
Tree Culture (*See* 963, Agriculture and Forestry)
Lighting Systems (*See* 371.2, Electric Lighting. Illumination, Exterior and 661.12, Gas Lighting. Illumination, Exterior)
155 **Grading, Drainage and Foundations**
.1 Grading
.11 Excavation

155	**Grading, Drainage and Foundations** (*Continued*)
.111	Earth
.112	Rock
.12	Embankments
.121	Earth
.122	Rock
.13	Subgrades
.14	Special Methods
.141	Swamps and Marshes
.142	Side-hill Grading
.2	Drainage
.21	Ditches
.22	Gutters
.23	Subdrains
.24	Culverts
	Catch Basins and Inlets (*See* 191.54)
.3	Foundations
.31	Natural
.32	Artificial
.321	Telford
.322	V-Drain
.323	Field Stone
.324	Broken Stone
.325	Gravel
.326	Cement-Concrete
156	**Materials**
.1	Earth, Sand and Clay
.11	Burned Clay
.2	Gravel
.3	Broken Stone
.4	Cement-Concrete
.5	Stone Blocks
.6	Wood Blocks
.61	Preservatives
.7	Brick
.8	Bituminous Materials
.81	Petroleums
.82	Asphalts
.83	Rock Asphalts
.84	Tars
.85	Tars and Asphalt Compounds
.9	Miscellaneous Materials
.91	Calcium Chloride
.92	Sulphite Liquor
.93	Shells
.94	Slags
.95	Clinkers
157	**Roads and Pavements**
.1	Earth and Sand-Clay
.2	Gravel
.3	Broken Stone
.4	Cement-Concrete
.5	Stone Block
.6	Wood Block
.7	Brick
.8	Bituminous Pavements
.81	Bituminous Macadam
.82	Bituminous Concrete
.821	One-Product Screening Plant
.822	Broken Stone and Sand
.823	Asphalt Block
.824	Topeka
.825	Bitulithic
.826	Warrenite
.827	Amiesite
.83	Sheet-Asphalt
.84	Rock Asphalt
.85	Miscellaneous
.851	Earth Mix
.852	Sand Mix
.853	Petrolithic
.9	Miscellaneous
.91	Slag
.92	Cobblestone
.93	Shell
.94	Clinker
.95	Wooden Plank

157 **Roads and Pavements** *(Continued)*
.96 Corduroy
.97 For Special Purposes
.971 Service Test Roads
.972 Race Tracks
.973 Bridle Paths
.974 Motor-Dromes
.975 Motor-Trackways
158 **Street Cleaning, Dust Prevention and Snow Removal**
.1 Street Cleaning
.11 Methods
.12 Reports (Arranged by Place)
.2 Dust Prevention
.3 Snow Removal
159 **Machinery and Tools**
.1 Grading Machinery
.11 Steam Shovels
.12 Road Drags
.13 Scrapers
.14 Graders
.15 Scarifiers
.2 Rollers and Tractors
.3 Crushing, Screening and Washing Machinery
.4 Mixing Machinery
.41 Cement-Concrete
.42 Bituminous Concrete
.43 Sheet Asphalt
.5 Distributing Machinery
.51 Watering Carts
.52 Bituminous Material Distributors
.6 Street Cleaning and Snow Removal Machinery
.61 Sweepers
.62 Squeegees
.63 Flushers
.64 Snow Plows
.7 Motor Trucks and Trailers
.8 Small Tools
.9 Miscellaneous
.91 Wagons
160 **Hydrology. Hydraulics. Dams**
161 **Hydrology. Water Resources**
.1 Physical Properties of Water
.11 Temperature
.12 Ice
.2 Rainfall and Run-off
.3 Stream Measurements
.4 Springs. Wells
.5 Underground Water
.6 Evaporation and Percolation
.7 Forest Influences
.9 Hydrology of Individual Localities (Arranged Geographically)
162 **Hydraulics**
.1 Instruments of Measurement
.11 Gauges (For Heights and Depths)
.12 Water-Flow Meters (For Volumes)
.13 Current-Velocity Meters
.14 Floats (For Velocities and Directions)
.15 Pitot Tubes } (For Pressures)
.16 Piezometers }
.19 Other Instruments
.2 Flow in Open Channels
.21 Back-Water
.22 Surges
.3 Flow over Weirs
.4 Flow through Orifices
.5 Flow through Short Tubes and Nozzles
.6 Flow through Pipes
.61 Water-Hammer
163 **Hydrostatics**
164 **Dams and Reservoirs. General**
.1 Foundations
.11 Exploration and Site
.12 Borings
.13 Seepage
.2 Design
.21 Profiles
.3 Stream Diversion

164 **Dams and Reservoirs. General** (*Continued*)
.4 Dam Accessories
.41 Gates
.42 Spillways
.43 Movable Crests
.44 Racks
165 **Fixed Dams**
.1 Earthen
.11 Hydraulic-Fill
.2 Rock-Fill
.21 Timber
.22 Steel
.3 Masonry
.31 Gravity
.311 Hollow
.32 Arched
.33 Reinforced Concrete
166 **Movable Dams** (Resting on River Beds)
.1 Trestle
.11 Needle
.12 Gates and Shutters
.13 Curtains
.2 Wickets
.3 Bear-Trap
.4 Rolling
.9 Other Types
Coffer-Dams (*See* 113.4)
167 **Reservoirs**
170 **Waterways**
General
.01 History
.02 Laws and Legislation
.021 Riparian Rights
.022 Water Rights
.023 Laws of Individual States or Provinces
.024 Laws of Individual Governments
.03 Statistics
.04 Costs and Estimates
.05 Contracts and Specifications
.06 Drawings
.07 Congresses
.08 Exhibitions
.09 Tests. Laboratories
.1 Water Transportation (Put here description of Special Kinds of Traffic (Grain Trade, etc.), also Traffic Statistics)
.7 Federal and State Relations
.71 Subsidies
.8 Economics
.81 Relation to Other Transportation
.82 Rates
.9 Inland Navigation Systems (Arranged by Country)
.91 Special Projects (*e. g.* Lakes and Gulf Waterways)
171 **Hydrographic Surveying** (Put actual surveys with the subject. For Stream Measurements, *See* 161.3, under Hydrology. Hydraulics. Dams)
172 **Excavation**
.1 Dredges and Dredging
.2 Rock Removal
173 **Coast Erosion and Protection**
.1 Groins, Spur Dikes, etc.
.2 Sea Walls (*See also* Retaining Walls, 114, under Structural Engineering)
.3 Reclamation of Tidal Lands
Breakwaters and Jetties (*See* 175.11 and 175.12)
175 **Harbors**
.01 History
.02 Laws and Legal Documents
.1 Protective Works
.11 Breakwaters
.12 Jetties
.13 Roadsteads and Anchorage
.14 Bulkheads. Dock and Quay Walls
.2 Development Works and Terminals
.21 Terminals
.22 Docks and Piers. Wharves (Includes enclosed docks used abroad)
.23 Freight Handling. Dock Machinery

175	**Harbors** (*Continued*)
.3	Operation
.31	Rates
.9	General Description of Individual Harbors (Arranged by Harbor)
176	**Canals**
	General
.01	History
.02	Laws and Legislation
.03	Statistics
.04	Costs and Estimates
.05	Contracts and Specifications
.06	Drawings
.07	Congresses
.08	Exhibitions
.09	Tests. Laboratories
.1	Location. Preliminary Investigations. Dimensions.
.11	Location of Individual Canals (Arranged by Name of Canal or Locality)
.2	Construction and Equipment
.21	Excavation
.22	Embankments
.23	Locks and Lock-Gates
.24	Inclines and Lifts
.25	Aqueducts and Siphons
.26	Enlargement
	Dams and Reservoirs (*See* 164)
.27	Traction
.271	Motive Power
.272	Boats
.273	Resistance of Water
.29	Of Individual Canals (Arranged by Name of Canal or Locality)
.3	Operation
.31	Management
.32	Finance
.321	Rates
.33	Freight
.34	Relations to Commerce and other Forms of Transportation
.35	Of Individual Canals (Arranged by Name of Canal or Locality)
.5	Company Reports
.6	State Reports
.7	Government Reports
.8	Federal and State Relations (General only. Special reports go with State and Government Reports)
.9	General Description of Canals (Arranged by Name of Canal or Locality)
177	**Rivers**
	River Hydraulics and Stream Measurements (*See* 160)
	Dredging (*See* 172)
.1	Regulation
.11	Storage and Regulation Reservoirs (Put here general discussion of the subject. For theory of design and construction, *See* 160)
.12	Channel Contraction
.13	Bank and Bed Protection
.131	Training Walls
.132	Spur-Dikes, Groins, Hurdles, etc.
.133	Ground Sills
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.22 Sulphuric Acid
.3 Alkalies
.4 Salts (For Common Table Salt, *See* 785, Salt, under Foods and Beverages)
712 **Paint, Varnish, etc.**
713 **Dyes. Colors. Inks**
714 **Distillation Products**
.1 Wood Distillation
.2 Coal Distillation (*See* Gas Engineering)
715 **Explosives. Matches** (*See also* 831, Explosives, under Military and Naval Science)
716 **Starch, Glucose, Gluten, etc.**

717 **Photographers' Supplies**
719 **Miscellaneous Organic Chemical Industries**
.1 Vegetable and Animal Oils
.2 Soap, Perfumery, etc.
.3 Glycerine
.4 Candles
.5 Waxes
.6 Gums and Resins
.7 Glue and Gelatine
.8 Fertilizers
720 **Ceramics. Stone, Clay, and Glass. Other Non-Metallic Mineral Products.**
721 **Ceramics**
.1 Pottery
.2 Stoneware. China
.3 Sanitary Ware
.4 Electrical Porcelain
.5 Brick
.6 Terra Cotta
.9 Other Clay Products
722 **Glass Manufacture**
723 **Cement, Lime, and Gypsum**
724 **Stone Products. Artificial Stone**
725 **Abrasives**
726 **Asphalt and Other Bitumens**
727 **Petroleum**
.1 Lubricants (*See also* 204, Lubrication, under Mechanical Engineering)
729 **Miscellaneous Non-Metallic Mineral Products**
.1 Asbestos
730 **Metal Manufactures. Machinery. Conveyances**
731 **Gold and Silver Ware. Jewelry**
732 **Iron and Steel Articles** (Not Classified Elsewhere)
733 **Wire Making**
734 **Hardware, Cutlery, and Tools**
735 **Implements. Instruments** (Not Classified Elsewhere)
.1 Typewriters
.2 Cash Registers
.3 Scales
.4 Meters
.5 Watches and Clocks
736 **Machinery.** (Not Classified Elsewhere)
737 **Locksmithing. Gunsmithing**
738 **Vehicles. Conveyances**
.1 Carriages, Wagons, and Sleighs
Automobiles and Motor Trucks (*See* 230)
.2 Bicycles
Manufactures of Miscellaneous Metals (Arranged Alphabetically, *e. g.*, Aluminum, Bronze, Copper, Tin, etc.)
740 **Lumbering. Wood Manufactures**
741 **Logging. Lumbering**
742 **Wood Working. Saw Mills**
743 **Furniture**
.1 House Furniture
.2 Office Furniture
.3 Pianos and Musical Instruments
.4 Mirrors
744 **Cooperage**
745 **Box Making**
746 **Caskets. Undertakers' Supplies**
747 **Pulp and Fiber Goods**
.1 Mats. Baskets
.2 Brooms
.3 Brushes
748 **Cork Industries**
749 **Miscellaneous Wooden Articles**
750 **Paper Making**
760 **Textiles**
.1 Design
.2 Processes
.21 Spinning
.22 Weaving
.23 Dyeing and Finishing
.3 Machinery
761 **Cotton Manufactures**
762 **Woolen Manufactures**

763 **Silk Manufactures**
764 **Flax, Hemp, and Jute**
765 **Dry Goods**
766 **Carpets and Rugs**
767 **Felt**
768 **Cordage and Twine**
769 **Oil Cloth. Window Shades**
770 **Leather Manufactures. Tanning**
771 **Boots and Shoes**
772 **Harnesses. Saddlery**
773 **Trunks. Traveling Goods**
774 **Fur**
779 **Miscellaneous Leather Goods**
780 **Foods and Beverages**
781 **Flour, Feed, and Other Cereal Products**
782 **Sugar and Molasses**
783 **Canning Industries**
784 **Chocolate and Cocoa**
785 **Salt**
786 **Groceries** (Not Otherwise Classified)
787 **Slaughtering. Meat Packing**
788 **Dairy Products**
.1 Milk and Cream
.2 Butter
.3 Cheese
.4 Ice Cream
.5 Casein
789 **Wines and Liquors**
.1 Brewing and Malting. Beer
.2 Wine Making
.3 Distilled Liquors
790 **Miscellaneous Industries** (Arranged Alphabetically, *e. g.,* Celluloid Products, Laundering, Rubber, Tobacco)

800 **MILITARY AND NAVAL SCIENCE**
General
.01 History
.02 Laws
.03 Statistics
.04 Costs and Estimates
.05 Contracts and Specifications
.06 Drawings
.07 Congresses
.09 Tests. Laboratories
810 **Military Science, General Works**
811 **Strategy and Tactics**
812 **Army Organization and Administration**
820 **Fortifications**
830 **Ordnance (Military and Naval)**
831 **Explosives**
832 **Military Ordnance**
833 **Naval Ordnance**
840 **Naval Architecture. Shipbuilding.**
.1 Buoyancy and Displacement
.2 Resistance
.3 Stability
.4 Strength of Ships
.5 Structural Details
.51 Water-tight Compartments
.6 Ventilation and Heating
.8 Measurement of Ships
.9 Miscellaneous Equipment of Ships
.91 Steering Apparatus
.92 Distillation of Salt Water
841 **Materials**
842 **Ship Propulsion. Propellers**
.1 Steam
.2 Electric
843 **Special Types of Vessels** (*See also* 870, War Vessels, under Naval Science)
.1 Ocean Liners
.2 River and Lake Boats
.3 Vessels for Freight only
.31 Oil Steamers. Tank Ships
.32 Colliers
.33 Ore Ships

843 **Special Types of Vessels** (*Continued*)
.4 Canal Boats. Barges
.5 Tugboats
.6 Motor Boats and Launches
.7 Yachts. Pleasure Boats
.8 Survey Boats
.9 Other Types
850 **Yards**
851 **Dry Docks**
860 **Navigation. Shipping**
861 **Nautical Astronomy**
862 **Nautical Instruments**
.1 Compass
.11 Magnetism of Ships and Compass Variation
863 **Seamanship**
864 **Aids to Navigation**
.1 Tide Tables
.2 Charts
.3 Sailing Directions
.4 Steam Lanes
.5 Signals and other Maritime Regulations. Prevention of Collisions
.51 Signals
.511 Lighthouses
.512 Light Ships
.513 Buoys
.514 Fog Horns
.52 Marking of Ships
.53 Flags and Pennants
.54 Steering and Sailing Rules
.55 Loading Rules
.6 Pilotage and Pilots
865 **Shipwrecks. Accidents**
.1 Life Saving
.2 Salvage
866 **Shipping. Merchant Marine**
.1 Directories, Blue Books, etc.
.2 Companies
.3 Rates
.4 Tolls
.5 Accounts
.9 Inspection
870 **Naval Science. War Vessels**
871 **Battle Ships and Cruisers**
872 **Torpedo Boats**
873 **Torpedoes**
874 **Submarines**
875 **Armor Plate**
876 **Armament**
880 **Naval Strategy and Tactics**
890 **Naval Organization and Administration**
891 **Registers.**
892 **Navy Customs. Rank of Officers**
893 **Personnel**
.1 Naval Militia
.2 Marines
894 **Education**
895 **Maintenance and Supplies**
896 **Hygiene**

900 OTHER SUBJECTS
Note: For subdivisions, use Dewey Classification.
910 **Philosophy**
920 **Religion**
930 **Sociology**
940 **Philology**
950 **Natural Science**
960 **Useful Arts** (Other than Engineering and Manufactures)
970 **Fine Arts**
980 **Literature**
990 **History**

APPENDIX C

AMENDMENTS TO THE CONSTITUTION, AMERICAN SOCIETY OF CIVIL ENGINEERS, ADOPTED, 1891–1917.

Oct. 3, 1894 (*Proceedings*, Vol. XX, p. 176.)	Art. II, Sec. 1, 2. Art. III, Sec. 6, 7. Art. IV, Sec. 1, 2, 3, 7.	Abolishes Grade of "Subscribers."	Yes, 194 No, 5
	Art. II, Sec. 8.	Corporate Members may become Fellows. Previous to this time the grade of Fellows was limited to "contributors to the permanent funds of the Society who may not be eligible for admission as Corporate Members."	Yes, 196 No, 3
	Art. III, Sec. 1.	Honorary Members "shall be elected only by the unanimous vote of the Board of Direction" instead of "by a unanimous vote of the Board of Direction and such Past-Presidents of the Society as continue to be members of the Society", etc., the Past-Presidents having become members of the Board of Direction on the adoption of the Constitution.	Yes, 196 No, 5
	Art. III, Sec. 2.	Applicants for membership must be endorsed by written communications from five Corporate Members before applications can be considered by the Board of Direction.	Yes, 193 No, 7
	Art. VI, Sec. 4.	This Amendment provided that the Secretary be a Corporate Member of the Society and be elected annually by the majority of the whole Board of Direction within twenty days after the Annual Meeting.	Yes, 191 No, 6
Mar. 3, 1895 (*Proceedings*, Vol. XXI, p. 85.)	Art. III, Sec. 3.	This Amendment required the Preliminary List of applicants to contain the names of "references" for Corporate Members and "endorsers" for Associates, Juniors, and Fellows, in order to make the meaning in Section 2 of this Article clearer.	Yes, 265 No, 14
	Art. III, Sec. 5.	This Amendment required that ballots on reconsideration of rejected applications for membership must be signed by the voters.	Yes, 224 No, 61
	Art. VII.	The territory occupied by the Membership was divided into Seven Geographical Districts, thus dividing the non-resident members of the Board of Direction equally among the six non-resident Districts. This amendment increased the elective members of the Nominating Committee from seven to fourteen, who, with the five living Past-Presidents, should nominate officers for the Society; made the term of each member two years; and empowered the Board of Direction to prescribe the mode of procedure in the election of members of the Nominating Committee.	Yes, 273 No, 12
	Art. VI, Sec. 5.	This Amendment provided for the appointment of an Assistant Secretary by the Board of Direction.	Yes, 273 No, 7

Oct. 6, 1897 (*Proceedings*, Vol. XXIII, p. 164.)	Art. V, Sec. 1.	This Amendment provided that only the "five latest living Past-Presidents who continue to be members" shall be members of the Board of Direction, instead of "all the living Past-Presidents", as previously provided. In the case of the election of Honorary Members, however, all the Past-Presidents shall be members of the Board of Direction.	Yes, 427 No, 47
Oct. 5, 1898 (*Proceedings*, Vol. XXIV, pp. 121, 167.)	Art. VI.	The office of Auditor was abolished and his duties were transferred to the Secretary; provision was made for auditing the accounts of the Society monthly, and the duties of the Finance Committee were widened, in order that the immediate supervision of the financial affairs of the Society might be put into the hands of such Committee.	Yes, 210 No, 1
Oct. 3, 1900 (*Proceedings*, Vol. XXVI, p. 216.)	Art VII, Sec. 2.	The time of appointing the Nominating Committee was changed from the Annual Convention to the Annual Meeting, and the time was fixed for the meeting of such Committee and its presentation to the Board of Direction of the nominations for officers to be elected at the next Annual Meeting.	Yes, 193 No, 53
Mar. 4, 1903 (*Proceedings*, Vol. XXIX, pp. 36, 100.)	Art. III, Sec. 2, 3.	This Amendment provided that hereafter the Board of Direction shall classify the applicant with his consent.	Yes, 401 No, 26
Oct. 7, 1903 (*Proceedings*, Vol. XXIX, pp. 195, 374.)	Art. II, Sec. 6, 7, 8.	By this Amendment provision was made for the omission of the clause, in the case of application for Junior membership, stating that the applicant intends to become or continue to be an engineer.	Yes, 537 No, 49
	Art. III.	By this Amendment all applications are to be sent out as applications for "admission" to the Society without classification into grades; power is given the Board of Direction to transfer persons from a lower to a higher grade; the number of negative votes for exclusion is raised from seven to twenty; and the reconsideration ballot (pink ballot) is abolished.	
Oct. 7, 1908 (*Proceedings*, Vol. XXXIV, p. 408.)	Art. III, Sec. 1, 3, 4.	The election and transfer of applicants in any grade is taken from the membership at large and given to the Board of Direction, the consequent changes in method of election are fixed, and the number of negative votes for exclusion is changed from "20 or more" to "3 or more".	Yes, 892 No, 317
	Art. VI, Sec. 12.	This Amendment confers on the Board of Direction the power of appointing a Special Committee when such appointment is approved by a business meeting of the Society; and, if it is necessary, in the opinion of the Board, that such Committee be appointed in order to accomplish the objects for which its appointment is requested.	Yes, 1123 No, 58
	Art. VII, Sec. 2.	Power is given the Board of Direction to fill any vacancies occurring in the Nominating Committee.	
Mar. 1, 1911 (*Proceedings*, Vol. XXXVII, p. 164.)	Art. IV,	A new Section (13) is added to Article IV by this Amendment, which provides for exemption from dues of Corporate Members and Associates who have reached the age of seventy years, and have paid dues as such for twenty-five years, and also of Corporate Members and Associates who have paid dues as such for thirty-five years.	Yes, 2229 No, 39

Oct. 2, 1912 (*Proceedings*, Vol. XXXVIII, p. 550.)	Art. VII,	By this Amendment the number constituting a quorum at a meeting of the Nominating Committee is fixed at ten; the time of meeting of the Nominating Committee is fixed to take place either at the Annual Convention or not later than July 15; provision is made for the organization of the Nominating Committee; "Official Nominees" and "Nomination by Declaration" are established; nomination by the Board of Direction is provided for, in case the Nominating Committee fails to act; and the time of closing the polls at the Annual Election is changed from noon to 9 A. M.	Yes, 680 No, 31
Mar. 3, 1915 (*Proceedings*, Vol. XLI, p. 150.)	Art. VII, Sec. 1.	The territory occupied by the membership is divided into Thirteen Districts instead of seven, as heretofore.	Yes, 1066 No, 83
	Art. VII, Sec. 2.	This Amendment provides for the method of electing the Nominating Committee from thirteen districts instead of seven.	

AMENDMENTS TO CONSTITUTION, AM. SOC. C. E.
REJECTED, 1891–1917.

Mar. 6, 1901 (*Proceedings*, Vol. XXVII, pp. 38, 82.)	Art. II, Sec. 5.	It was proposed by this Amendment to add a clause to Section 5, Article II in order to allow the Board of Direction to transfer any Junior elected prior to the adoption of the Constitution in 1891 to the grade of Associate.	Yes, 282 No, 166
Mar. 5, 1902 (*Proceedings*, Vol. XXVIII, pp. 35, 98.)	Art. III,	This Amendment, if adopted, would have placed the election of all members in the hands of the Board of Direction.	Yes, 343 No, 257
Mar. 6, 1907 (*Proceedings*, Vol. XXXIII, pp. 71, 152.)	Art. II.	It was proposed by this Amendment to raise the standard of membership in the Society by raising the qualifications for admission to the various grades.	Yes, 429 No, 847
	Art. III, Sec. 2.	This Amendment related to applications of engineers not resident in North America and provided that the applicant must possess the necessary qualifications for membership before he is recommended for election to the Society.	
Mar. 3, 1909 (*Proceedings*, Vol. XXXV, p. 160.)	Art. III, Sec. 4.	This Amendment provided that negative votes equal to 1%, or the whole number nearest to 1%, of the total Corporate Membership at the time of voting shall exclude from membership. This Amendment was nullified by the Amendment adopted on Oct. 7th, 1908, and was, therefore, defeated.	Yes, 247 No, 762
Mar. 4, 1914 (*Proceedings*, Vol. XL, p. 176.)	Art. VII, Sec. 1, 2.	(A) By this Amendment, it was proposed to divide the territory occupied by the membership into Thirteen Districts; it also provided for the procedure in appointing the Nominating Committee from such Districts at the Annual Meeting.	Yes, 1494 No, 1628
	Art. VII,	(B) This Amendment also provided for dividing the territory occupied by the membership into Thirteen Districts and the method of procedure of electing the Nominating Committee by ballot to be counted by the Board of Direction and announced to the Annual Meeting.	Yes, 1550 No, 1612

Art. V, Sec. 1, 2.	(C) By this Amendment, it was proposed to change the status of the Secretary by removing him as a member of the Board of Direction. It also defined the terms of officers elected by the Society.	Yes, 1343 No, 1828
Art. VI, Sec. 4, 6.	The changes proposed in this Amendment relate to the method of electing the Secretary by the Board of Direction and would have given the Board of Direction power to determine the salaries to be paid to the Secretary and Treasurer.	

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

PAPERS AND DISCUSSIONS

FINAL REPORT OF THE SPECIAL COMMITTEE ON MATERIALS FOR ROAD CONSTRUCTION AND ON STANDARDS FOR THEIR TEST AND USE*

To the American Society of Civil Engineers,

Gentlemen:

Your Special Committee on "Materials for Road Construction and on Standards for Their Test and Use" herewith respectfully submits a Final Report covering its work from the date of its appointment by the Board of Direction.

Through the courtesy of the Board, successful meetings have been held in the Society House, for one or two days subsequent to each Annual Meeting of the Society, for the discussion of matters of interest connected with the work of the Committee. These meetings have been well attended, usually by more than 200 at a session, by highway authorities outside of the Society as well as by members of the Society itself. It is felt that valuable, as well as authoritative, information and criticism has thus been secured to the Profession as well as to the Committee.

Your Committee has rendered progress reports annually since 1911, and in the accompanying report has given careful consideration to its earlier conclusions and the criticism and discussions submitted in connection therewith.

The Committee has been made aware of a widening of the interest in highway work among members of the Society and others not experienced or even thoroughly trained in highway engineering. In the accompanying report it has been deemed wise, for the benefit of this new and wider interest, as well as for the purpose of making this Final Report comprehensive, to include conclusions or statements that may seem primary or trite to highway experts. Further, in order to render many of the earlier and present conclusions of the Committee

* To be presented to the Annual Meeting, January 16th, 1918.

more intelligible to the wider interest mentioned, as well as to demonstrate their practical application in many cases, and to suggest the variables still remaining to be determined in specific instances, it has been thought wise to submit in this report principles underlying the drafts of specifications.

Your Committee believes that competent highway engineers may meet successfully the demands of any particular case by following these principles and eliminating the variables, necessarily left therein in order to express conclusions of general application, after proper consideration of the local factors affecting them.

However, it does not believe that these conclusions as to specifications will offset a serious lack of knowledge or experience in highway engineering or furnish a discriminating ability otherwise lacking. It also wishes to warn against any attempt to reduce the principles for specifications to the establishment of "the one best pavement" for any conditions.

One of the main activities of the Committee since its foundation has been toward securing accuracy and uniformity in the recording of data connected with highway work. To this end the Committee has suggested tests and analyses of materials in detail, records of traffic, costs, construction and maintenance details, and definitions of terms used with peculiar meanings in highway work. When it began its work in 1909, the status of affairs in these respects was found to be deplorable. Since then the work of this Committee and of committees on similar lines from other societies, as well as the work of some individuals, has materially improved the situation. Your Committee has made some suggestions which have been quite widely adopted. On the other hand, it has not hesitated to endorse the conclusions of others where such action seemed proper, hoping thereby to aid materially in the accomplishment of that desirable uniformity mentioned above. It hopes that its conclusions may be generally approved, even at the sacrifice of some minor differences of opinion, in order that uniformity in these matters, with resulting benefit, may be secured at the earliest possible date. The Forms, Analyses, and Test Details will be found in the Appendices hereto attached.

In its previous reports, your Committee has suggested a number of matters worthy of investigation. Sufficient time has not yet been available to the Committee for reaching satisfactory conclusions on all these points, and it has seemed to be a grave question whether or not it would be able, with the facilities at its disposal, to reach such conclusions within a reasonable period. Your Committee, therefore, invites the attention of all those interested to the following list of investigations it thinks worthy of prosecution, and expresses the hope that some authoritative data, which will throw light on these points, may in some way be secured at an early date:

The effect of various kinds of earth or soil on bituminous carpets under known conditions of traffic;

The efficiency of the use of asphaltic oils for surface treatments on gravel and broken stone roadways;

The standardization of, and determination and expression of the consistency of, bituminous materials which preferably may permit of its use on all the wide range of such materials, from liquid to solid, and of the inter-relation existing in certain, if not all, cases;

The maximum and minimum quantities of free carbon that can be successfully allowed in tars under known conditions of other constituents of the tar, of climate, of traffic, and of methods of use;

Methods of determining the adhesive strength of bituminous cements;

The advisability of incorporating in bituminous pavements, built by the penetration method, bituminous material in excess of the minimum quantity necessary for the production of satisfactory results;

The determination of the amount of internal wear of the materials comprising a macadam roadway under known conditions of traffic, and the effects on the internal wear of increasing or diminishing the size of the stone in the courses;

The determination of the causes of cracks in concrete and brick roadways, and the best methods to be used to reduce such cracks to a minimum;

Rolling or other methods calculated to increase the surface density of cement-concrete pavements;

The relative merits of one-course and two-course plain cement-concrete pavements, and plain and reinforced cement-concrete pavements;

The relative efficiency of bituminous and cement fillers for block pavements;

The determination of the quality—which might be called its hardness or resiliency—of wood used for paving purposes;

The continuance of traffic census, and study of the results obtained, to determine the effect of motor traffic on various roadway surfaces, and especially with reference to the speed of the vehicle, and the establishment and expression of any relation between the traffic and the life or cost of any pavement;

The further study of suitable joints for block pavements on steep grades;

Proper methods of sampling highway materials.

Many points are as yet undetermined, and seem improbable of early settlement. Conclusions, in many instances, will be hastened by a prompt agreement among investigators that uniformity is fully as necessary as accuracy in their work, and that the use of methods of analysis conforming to those carefully specified in this report will produce better and quicker results than a wide use of various methods, more or less accurate in some details, but not consonant with those of the Committee. The conclusions of your Committee have been reached and expressed after careful consideration of the work of committees of other societies and with intimate knowledge of the details of that work through the membership on these other committees of members of this Committee.

Your Committee is impressed by the importance of the factor of costs, both as to construction and maintenance of highways, and of the need for comparable records of such data. The present situation in this respect is unfortunate, there being found available but few records of costs uniformly or logically compiled. Hence comparisons are difficult, if not impossible. Your Committee, therefore, invites especial attention to its conclusions and forms in the matters of costs. The same may be said concerning traffic records. The propriety of a material or a form of highway construction is often determined by its cost under known traffic conditions. To reduce both cost and traffic data in different cases for purposes of comparison requires that they be expressed in some uniform, definite, and intelligible manner.

Your Committee submits the specific conclusions it has reached, as follows:

General Conclusions.

Selection of Roadway Surface.—The Committee believes that, with the development of highway work, it should be constantly more apparent that one of the greatest problems to be solved by highway engineers is the proper selection of the particular material and form of construction to be used which will most efficiently meet the conditions of any particular case, and that progress will be hastened by complete recognition of this fact. It recommends that the selection of the kind of crust or pavement be based on the following factors, the special value of which may be estimated in each case under the local conditions of traffic, surroundings, climatic conditions, and physical and financial resources, both as to construction and maintenance, with proper regard for probable or possible changes in these circumstances: First cost, maintenance cost, annual cost, ease of maintenance, durability, cleanliness, tractive resistance, slipperiness, favorableness to travel, sanitariness, noiselessness, and appearance.

Traffic.—Your Committee desires to emphasize the fact that experience has demonstrated the value of a traffic census taken both

preliminary and subsequent to the construction of a highway. The traffic census should be considered one of the most important variable factors in the solution of the problem of the selection of that type of construction best suited to local conditions, considered from the standpoints of both economy and efficiency. In connection with the census returns on any highway, should be considered the traffic on cross and parallel highways and the effect of improvement of these highways on the traffic of the highway under construction. The bald return of a traffic census, however, should not be the sole basis of the selection of the type of construction, but should be considered a guide in determining the value of the type to be adopted. In considering the effect of traffic and its relation to the design and cost of maintenance, it is necessary to take into account the speed as well as the weight of the vehicles.

Costs.—The Committee recommends that highway departments, in making their reports, adopt a tabular form similar to the one submitted in Appendix A.

Grades.—A choice of the material, or methods of using a particular material, may be affected by the grades as fixed. Certain materials or results of using materials for highway surfacings will be unsatisfactory outside of certain limits of grades. Conservative practice fixes the maximum limits for satisfactory results with grades as follows:

Kind of Roadway.		Maximum Grade.
Asphalt block		8.0%
Bituminous surfaces		6.0%
Bituminous concrete		8.0%
Bituminous macadam		8.0%
Brick	cement filler	6.0%
	bituminous filler	12.0%
	"Hillside" block	15.0%
Broken stone		12.0%
Cement-concrete		8.0%
Gravel		12.0%
Sheet-asphalt		5.0%
Stone block	cement filler	9.0%
	bituminous filler	15.0%
Wood block		4.0%

The minimum grades allowable will depend on local conditions as to climate, type of construction, character and amount of traffic, conditions of underlying and adjacent soil, and such other circumstances as affect drainage. Except for roadways on fills, where the outside edges of the surfaces of the shoulders are at least 2 ft. above the level of the adjacent ground or water level, or except in cases

where the roadway is laid over sand of such a character that it never becomes water-logged, a longitudinal grade for the roadway of less than one-half of 1% should not be used for roads.

On streets where the smoothness and evenness of the roadway surface may be confidently expected to be the greatest, and where conformity to the proposed elevations of surface is more carefully sought and more accurately possible, a minimum grade of as low as one-quarter of 1% has sometimes given satisfactory results in an emergency; but a minimum of one-half of 1% would be better as an established standard.

Widths.—The width of the roadway to be built will be determined largely by local circumstances, but, in view of the recent, constant, and rapid increase of traffic on highways, both in number of vehicles and in size of loads, it will be in the interest of economy for designs of highways to be made with proper consideration of further increase.

Where motor traffic forms a considerable proportion of the total traffic likely to use a highway, the unit width of traffic lines to be considered is 9 or 10 ft., instead of 7 or 8 ft., as heretofore, because of greater clearance required for the safe passing of the units of such traffic.

Where bituminous pavements are laid, the edges need protection, and a sudden transition from the pavement to any soft shoulder material should be avoided by means of extra width, or of cement-concrete or other edges, or such reinforcement of the shoulder material as may be necessary.

The width of roadways of rigid material, such as cement-concrete or vitrified block, should be at least equal to what would be prescribed under local conditions for a less rigid surfacing. The great difference between the firmness of a rigid roadway surfacing and of material frequently available for the shoulders thereto, often makes it necessary for safety and convenience of traffic, as well as for economy of maintenance, that the rigid surfacing shall be built wider than would answer for a more flexible surfacing, such as water-bound macadam, for instance, under the same local conditions. For single-track roadways, the width of the pavement should not be less than 10 ft., and for two lines of traffic, it should not be less than 18 ft., unless exceptionally durable shoulders are provided. In a street or alley, the width will ordinarily be determined by the necessary location of the curb.

Too narrow a width of roadway encourages, if it does not compel, concentration of traffic to such an extent as frequently to make unfair demands on what would otherwise be a suitable and efficient material for the surfacing. This may be especially noticeable at abrupt changes in the lines of the highway, where any tendency toward the improper concentration of traffic into too narrow areas should be avoided, as

far as possible, by such adjustment or separation of lines, and adjustment of width, of crown, or of slope, of the roadway surfacing as will keep the strains of the surfacing material within reasonable limits.

Too great a width for the roadway surfacing is as unwise in many ways as too little. Excessive width not only results in unnecessary first cost and interest charges, but also in needlessly increased maintenance and cleaning costs. Further, especially in the cases of those pavements where at least a minimum amount of travel is needed to preserve the surface in good condition, an excess of width may result in the development of areas from which disintegration of the whole pavement may rapidly spread.

Thickness.—In determining the thickness of any road crust or pavement, there must be consideration of the character of the foundation and of the weight of the vehicles to be supported. Although the general practice has been, too often, perhaps, to use mass, for the sake of safety in the preparation of the pavement, it now appears evident that some waste has been incurred in this direction, and that a more scientific determination of the thickness is possible without sacrifice of safety and yet with economy. However, in view of the recent, constant, and rapid increase of the weight, and consequently of the strains, caused by the traffic, it will be in the interests of economy for all designs of highways to be made with proper consideration of further increases.

In considering the character and capabilities of the foundation relative to the forces coming thereon through a pavement, the condition of the foundation under the most adverse conditions likely to exist for withstanding the forces, and the traffic stresses present and probable, and the character of the surfacing itself, should be taken into account. An absorbent sub-grade material likely to become soaked with water so as to weaken its supporting power may require a thicker slab, or even the addition of an artificial foundation, in order to disperse properly the stresses from the surface of the pavement. For instance, if laid on a strong sub-grade and one likely to remain always in good condition, the minimum thickness of 6 in. for a broken stone roadway will be sufficient, and possibly even 4 in. will be enough where the vehicles passing over the road are comparatively light, that is, of 1 or 2 tons on four wheels. With a concrete slab, ordinarily, from 4 to 8 in. in thickness will suffice, though, in order to prevent the possibility of a sudden rupture of the slab on some sub-grades by an exceptional load, a uniform thickness above the minimum may be wise. Again, mass—that is, unusual thickness—may in some cases be desirable with the use of a minimum of cement, not only for reasons of economy, but also for the purpose of avoiding the ill effects of frost and possibly of preparing for future developments that may seem probable. A thickness for a concrete slab in excess of 8 in. should

be determined upon only after thoroughly considering the possibilities of meeting the necessities of the case by other means, such as improving the sub-grade.

Although the distribution by the road crust or pavement of the traffic stresses through it to the foundation is not at present within the possibilities of calculation in the case of all types of pavements, progress has been, and is being, made along these lines. Logical and fairly accurate formulas have been developed in the case of broken stone road crusts, and studies productive of some results have been made in the case of some other surfacings. Also, the bearing power of soils of different kinds has been given considerable study. These studies should be encouraged and carried on, so that the thickness of the road crust or pavement for any type may be rationally determined.

Uniformity of thickness for the surfacing is made for the purpose of conducing toward uniformity of wear. Variations in the thickness of such surfacings as sheet-asphalt, for instance, invariably result in non-uniformity of wear, with a resulting increase of expense for satisfactory maintenance; and it is believed that the same cause is responsible for many of the difficulties experienced in the maintenance of block pavements of various types. The development of depressions in the surfaces of these pavements and the deterioration of areas of the pavement seem to be explained by the irregularities in the thickness of the surfacing or of the cushion or foundation under the surfacing over such areas, which irregularities produce unequal settlement or decided differences in rigidity.

The thickness of the pavement or surfacing, of course, will be dependent largely on its type. Approved practice establishes the limits shown in Table 1 for the extremes of thickness for the various layers of the pavement or road crust.

TABLE 1.

Kind of roadway.	Thickness of artificial foundation* (ordinary), in inches.	Thickness of sand cushion or binder course, in inches.	Thickness of wearing course, in inches.
Asphalt block	5 to 8	....	2 to 3½
Bituminous surfaces	4 to 8	....	¼ to ½
Bituminous concrete	3 to 8	....	1½ to 3
Bituminous macadam	3 to 8	....	2 to 3
Brick	4 to 8	¾ to 1½	3 to 4
Broken stone	3 to 8	....	2 to 3
Cement-concrete { One course	....	....	5 to 8
Cement-concrete { Two course	4 to 8	....	2
Gravel	4 to 8	....	2 to 4
Sheet asphalt	5 to 8	1 to 1½	1½ to 2
Stone block	5 to 12	1 to 2	2½ to 5
Wood block	5 to 8	0 to ½	3½ to 4

* Not including extraordinary provisions such as V-drains or "sub-base" courses.

Drainage.—The use of any form of pavement or road crust, whether bituminous or non-bituminous, does not relieve the necessity for proper drainage in every case. It is not only necessary to provide for such under-drainage as will place and keep the sub-grade in a condition satisfactorily free from moisture and in a state of suitable efficiency, but it is also necessary to provide and to preserve economically such provisions for surface drainage as will, with the provisions for under-drainage, insure these results fairly permanently. Storm-water coming to the roadway must be carried quickly and rapidly away from it by automatic arrangements to the natural watercourses where it can be finally disposed of. The arrangements referred to and so made, such as inlets, ditches, gutters, and culverts, should be designed and placed so as give the least possible offense to the users of the roadway and the abuttors, and yet be built so as to preserve their integrity and efficiency with the least need for attention and expense, under even the most persistently adverse natural conditions. A proper longitudinal grade for ditches and gutters is particularly important, in order that the ill effects of standing water may be avoided. A proper cross-section for ditches is also important, in order that the water may not become obstructed by the sliding in of the sides.

The under-drainage of the roadbed, where a cement-concrete roadway or an artificial foundation of cement-concrete is to be provided, should be at least as good as that which would be required in most cases of other surfaces, because the rigidity of the cement-concrete slab does not permit it to adapt itself—as is the case, for instance, with such a surface as macadam—without injury to changes in the sub-grade resulting from defective drainage.

As related to drainage, the matter of the crown of the roadway is particularly important. The ideal roadway surface would be flat in cross-section were it not for the necessity for the removal of surface water to the channels where it must be most conveniently carried along. Crowning the roadway tends to concentrate the traffic on the ridge, where it is then most comfortable for the travelers, and the amount of crown which will result in this concentration on the ridge varies with the type of pavement. Also, the rate of crown necessary for the proper removal of storm-water to the gutters or ditches varies with the type and with the provisions to be made for the cleaning and the upkeep of the roadway surface. In the general practice, the amount of crown for the shoulders of an uncurbed roadway has usually been a cross-slope of 1 in. per ft., the shoulders being of the natural earthy material, and this rate is to be recommended for shoulders, except in special cases.

With pavements inclined to be slippery under certain conditions the crown should be reduced to the lowest possible minimum consistent with surface drainage; and, where the longitudinal grade is sufficient

to allow the water to run off freely, the crown should be very flat—not exceeding 3 in. in a roadway width of 30 ft.

For the various roadway surfacings, the practice generally observed and to be recommended is stated in Table 2.

TABLE 2.

Kind of roadway.	Maximum.	Minimum.
Asphalt block	¼ inch to the foot.	⅛ inch to the foot.
Bituminous surfaces	½ " " " "	¼ " " " "
Bituminous concrete	½ " " " "	¼ " " " "
Bituminous macadam	½ " " " "	¼ " " " "
Brick pavements	⅜ " " " "	⅛ " " " "
Broken stone	¾ " " " "	½ " " " "
Cement-concrete	⅜ " " " "	¼ " " " "
Gravel	1 " " " "	½ " " " "
Sheet-asphalt	¼ " " " "	⅛ " " " "
Stone block	½ " " " "	¼ " " " "
Wood block	¼ " " " "	⅛ " " " "

Concave pavements of cement-concrete, vitrified block, or stone block, may frequently be found advantageous for alleys, and, in such cases, the same rates of slopes in cross-section as those given in Table 2 should govern.

Sub-Grade.—The use of any form of pavement or road crust does not relieve the necessity for the construction of a well-drained, thoroughly compacted, homogeneous, and stable sub-grade in every case. Indeed, such improvement of the highway generally attracts heavier traffic, and thus increases the stresses in the sub-grade. Even when an artificial foundation is to be constructed on the sub-grade, proper care should be taken in its preparation in order that the greatest economy may be had in the design and expense for the artificial foundation, and, generally speaking, at least, the higher the type and the more expensive the artificial foundation, the greater care should be had to develop to the utmost the possibilities of the sub-grade. Uniformity in the composition and compaction, as well as evenness of its surface, is far more important than has been generally considered necessary, and permanence of all the desirable qualities in the sub-grade is equally important.

When an artificial foundation or a cement-concrete pavement is used, the sub-grade should be as carefully prepared, rolled, and compacted, as for any other roadway surfacing, and should be made to conform accurately to the proper lines and grades. If necessary, the sustaining powers of the natural foundation should be reinforced and the strains in it thereby further distributed by the interposition between it and the cement-concrete slab of an artificial foundation consisting of a layer, or layers, of sand, gravel, broken stone, or similar material. Reliance should not be placed on the concrete, if used, for bridging soft, spongy, or unyielding spots; all vegetable or

perishable matter should be removed from the sub-grade, and such other material substituted as will insure a thoroughly compacted and homogeneous sub-grade for the concrete. It may be necessary, for proper compaction, to use water in connection with rolling, and, in any event, cement-concrete should not be deposited on a dry, absorbent sub-grade.

Every precaution should be taken to prevent a disturbance of the sub-grade after it is completed and until the next layer is deposited.

When a cement-concrete or other rigid pavement is to be constructed over an old surfacing composed of gravel or broken stone macadam, the old pavement should be loosened, spread to the full width of the new pavement, and then thoroughly compacted, filling the interstices with fine material and re-rolling until a dense sub-grade is obtained.

Artificial Foundations.—Where the character of the traffic justifies the use of an artificial surfacing, it also demands a correspondingly strong foundation. Whether or not an artificial foundation shall be supplied will depend on the local conditions, but, in the selection of the materials and the methods of construction of the artificial foundation, every consideration should first be given to the possibilities for securing the greatest efficiency from the natural foundation. Economy in reference to the roadway will be had from the proper choice of the various materials available for artificial foundations, such as sand, gravel, broken stone, and concrete.

Owing to the inherent lack of elasticity in brick or cement-concrete pavements, it is especially necessary that the surface of the wearing course of such pavements shall be built and remain smooth and true to contour, for the sake of ease of traction, comfortable riding, and integrity of the surface, particularly where a cement filler for the joints is used. Special care, therefore, should be taken in these cases to provide foundations of ample strength and permanence to this end. Stone block pavements should generally, and wood block pavements should always, be laid on a cement-concrete foundation, but, in case of temporary paving with stone blocks, any type of stable foundation may be used.

Local conditions, occasionally, may justify the omission of an artificial foundation for brick pavements where the natural foundation material can be satisfactorily prepared and a reasonable permanence then expected from it, where a relatively low first cost of the brick prevails and where light traffic only is to be expected.

Any artificial foundation should be of a substantial thickness and be properly consolidated; and should be rendered so compact that further settlement or displacement will be avoided to the utmost.

The most usual proportions for a cement-concrete foundation have been one part cement, three parts fine aggregate, and six parts coarse

aggregate. This standard, however, is empirical rather than scientific, and a more rational proportion should be developed according to the needs and facilities of each case. Sometimes it may be desirable to increase the mass at the expense of unit strength, or to increase the mass for the sake of economy in the more expensive material. In mixing, placing, and completing a cement-concrete foundation, the principles expressed under the head of "Cement-Concrete Pavements" apply, and reference thereto should be had.

Materials.—Having determined the characteristics desirable for the materials to be used in any highway work, their description in the specifications should be concise, clear, and exact. Although, in some cases, it may not be possible to designate precisely the characteristics desired, it will be possible to specify that these qualities shall lie between certain limits, thus giving a reasonable tolerance to the determination of the quality by test as well as avoiding uncertainty as to whether or not a quality in this respect of a material offered is suitable. The description of a material by means of a trade name is permissible only in most unusual cases, and such a description as "equal to" another similar material should never be used. Qualities of a material or methods of its use should not be left "to the satisfaction of the engineer" or "as determined by, or in the opinion of, the engineer."

Specific tests and such description of the methods of performing each test as will leave no room for doubt as to whether the results of the tests come within the limits of tolerance should always be expressed, either in detail or by reference to the standards of some reputable authorities. The tests and methods of performing them, to be found in detail in Appendices B and C, are recommended for this purpose.

Whenever comprehensive specifications are to be prepared so as to admit a variety of types of bituminous materials, separate specifications, as may be necessary, should be written for each type.

Joints.—For the ordinary joints in block pavements, the materials and methods of filling should be selected so as to produce, not only a surface which will retain to the utmost its imperviousness and the stability of the blocks themselves in place, but, also, they should, as far as practicable, conduce toward evenness of wear of the surface of the pavement. If the blocks are resistant to abrasion, but are inclined to round off at the edges of the upper surface under traffic, such filling of the joints is desirable as will reduce rounding off at the joints. On steep grades, where some roughness of surface may be desirable for the sake of affording better foothold for animals, some openness at the top of the joint is desirable, and the softer joint fillers, less resistant to wear, may be preferred.

Joint fillers naturally are divided into two main classes—the hard cement-mortar filler and the soft bituminous filler. As it is desirable

to secure water-proof pavements, sand alone should never be used as a joint filler. Though the use of sand joints may occasionally appear to be justified in the interests of economy, it will generally be found unwise to use such joints when some relatively slight additional first cost will result in appreciably prolonging the life of the pavement. A bituminous filler may be preferred to a cement-grout filler on account of the lower cost of street opening repairs, the better foothold provided for horses, and the securing of a more elastic and less noisy pavement.

Cement-mortar joints when properly made will conduce to integrity of the surface. The proportions of sand, cement, and water will be affected by local conditions. To insure the best results, a 1:1 mix of sand and cement is recommended. Great care is necessary in mixing and applying the mortar or grout. Uniformity in the cement grout, and especially skill and care in its application, are essential to success. To insure uniformity, there should be a constant agitation of the mix, up to the moment of its application, and no more water than is necessary for proper fluidity should be used. Ample time should always be allowed for the grout to set thoroughly before the traffic is admitted to the roadway.

With bituminous joint fillers, care must be taken to select materials which will not be too brittle in cold weather and so chip out from the joints under traffic, and which will not be so soft in hot weather as to flow out of the joints between the blocks. It is believed, although not yet generally admitted as having been actually proved by experience, that the use of a bituminous mastic for joint filling would be an improvement over the customary practice of using bituminous material alone for this purpose. One of the great difficulties with bituminous fillers of any kind will be that of properly filling the joints between the blocks, and great care must be taken to insure this result.

Expansion-Contraction Joints.—Joints at intervals across certain types of pavements, such as brick and cement-concrete, as well as along the curbs, have been used in order to compensate for a more or less unavoidable movement of the pavement slab, which takes place under different conditions of moisture, or temperature of the air. In cases where expansion-contraction joints across the roadway at intervals are decided on, the Committee recommends the use of bituminous material and the abandonment of all forms of the so-called "armored" joints, because of the smaller interruption to the homogeneity of the roadway surface thus secured.

Cushion Courses.—The function of a cushion between the brick or block on an artificial foundation of cement-concrete is to allow for irregularities in the surface of the concrete and in the depths of the brick or block, and to give resiliency to the wearing course. If the surface of the concrete foundation is made true to the adopted cross-section, and as the variation in depths of the brick or block decreases,

the thickness of a sand cushion may be correspondingly decreased. The thickness should never be greater than that necessary to compensate for the unevennesses referred to plus such a thickness in the layer as will enable the latter to perform satisfactorily its function as a cushion, that is to say, about ½ in. for this latter addition. If the surface of the concrete is truly parallel with the finished pavement, and if the variation in the depths of the bricks or blocks does not exceed ⅛ in., the thickness of a sand cushion can safely be reduced to ¾ in. The material which has generally been used for the cushion course is sand, but engineers, recently, have been considering the advisability of using Portland cement mortar instead, the mortar being spread either on the surface of the concrete foundation after the latter has been completed, or being laid as a part of the foundation itself. The objection made by some to mortar is that it makes too solid a base for the blocks and gives no resiliency. This at present seems to be a moot question. The substitution of the mortar results in a monolithic structure, perhaps of greater strength, but less capable of absorbing shock without injury than is the case where a sand or bituminous cushion is introduced between the wearing surface and the foundation, especially where the joints are filled with a bituminous filler.

The sand for a cushion course may be slightly more loamy than that permissible for safety in mortar or cement concrete work. This excess loaminess may be advantageous in offering the sand a greater ability to resist displacement; but it should not be so loamy or fine as to prevent proper spreading and compaction, or to afford such a degree of capillarity as will result in frost action.

A bituminous mixture composed of sand, stone screenings, or possibly wood fiber, and a bituminous cement would probably be more satisfactory for many reasons than either the mortar course or the sand alone, and such a bituminous mixture needs to be only from ¼ to ½ in. in thickness, provided the surface of the concrete foundation is made sufficiently smooth and regular in contour.

Finishing of Surface.—An objectionable slipperiness of many pavements may be decreased or prevented by proper precautions during construction or by proper treatment thereafter. The length of time that a finished pavement should be closed to traffic in order to season properly before use varies from a few hours to several days, dependent on the character of the material and methods used and on climatic and other local conditions. Pavements in which Portland cement is used for filling the joints or in the mass of the surfacing itself should seldom, if ever, be closed for less than 2 weeks after completion. Although the plans and specifications usually call for the surface to be finished to definite cross-sections, grades, or contours shown on the

plans, questions frequently arise under contracts as to the importance of variations in exactness of compliance in the finished surface.

The Committee has had a number of observations made, from the results of which it is convinced that, in a newly completed pavement, the variations from a straight-edge or template, 8 ft. in length, should not exceed ½ in. for asphalt block, bituminous concrete, brick, cement-concrete, sheet-asphalt, and wood block pavements, and ¾ in. for broken stone roads, and bituminous macadam and stone block pavements.

Manhole Covers and Street-Car Tracks in Roadway Surfaces.—Uniformity in the roadway surface being essential for minimum wear and expense of maintenance, anything, such as manhole covers or street-car tracks, introducing an element of non-uniformity into the surface, should be counteracted as far as possible whenever surfaces of different degrees of hardness adjoin in a pavement. The traffic coming from the harder to the softer surface naturally causes abnormal wear on the latter.

Practically all manhole covers are laid on rigid masonry, so that, unless some special treatment is given to the pavement, there is apt to be undue wear adjacent to such structures. Where the pavement is of stone block, wood block, or brick, the difference in hardness between them and the manhole heads is not so great as with other paving materials, so that, when these pavements are used, they can be laid flush with the manhole covers, provided extra care is taken with the foundation immediately adjacent to the heads, to prevent any possible settlement. In the case of macadam or bituminous pavements, a different treatment should be used. In such cases the pavement should be laid about ½ in. above the manhole head. This will prevent the abnormal wear caused by the pounding action of the wheels of vehicles, which would occur if the pavement surface was any distance below that of the manhole head, a condition which must exist if the pavement is laid level with such heads, as it is not possible to compact the pavement so that it will not compress under the traffic, forming depressions before those actually caused by wear begin.

In the case of car tracks in streets, modern construction is such that the tracks are nearly rigid, although this condition does not exist in all cities at present. Where it does exist, however, the pavement should be laid, as in the case of manhole heads, somewhat above the level of the rail. The Committee, however, does not believe that in any case a bituminous pavement should be laid between the tracks or between the rails of the tracks. When car tracks are laid in roads the construction is not generally as nearly rigid as in streets, and the rails are usually of the T form. The tracks in such cases are often laid at the side of the road, rather than in the center, as is customary in streets. In the case of macadam or bituminous roadways, and when the rails are in the center, it would be advisable to lay stone blocks or

brick for a width of at least 18 in. adjacent to the rails; when the rails are at the side and the railroad area does not form a part of the roadway proper, loose broken stone or gravel may be substituted for the stone blocks or brick.

Asphalt Block Pavements.*

General.—Specifications for asphalt block pavement should cover thoroughly the several components of the bituminous concrete used, the manufacture of the blocks, the blocks *per se,* and the details of construction of the pavement.

Materials.—Experience has demonstrated that the blocks should be composed of asphalt cement, crushed trap rock or equally hard and tough material, and mineral dust. All particles of the trap rock should pass a ¼-in. screen, and the mineral dust or filler should consist of powdered limestone. The bitumen content of the blocks should be between 6.5 and 8.5%, depending on the grading of the mineral aggregate and the method of manufacture. The specifications should contain specific requirements with reference to the asphalt cement, filler, and the grading of the mineral aggregate, which latter should be similar to the following:

Passing 200-mesh sieve	20 to 35 per cent.
Passing 80-mesh sieve and retained on 200-mesh sieve	7 " 15 " "
Passing 20-mesh sieve and retained on 80-mesh sieve	12 " 30 " "
Passing ¼-in. screen and retained on 20-mesh sieve	30 " 50 " "
Retained on ¼-in. screen	0 " "

The specifications should also cover the specific gravity of dry blocks, which should not be less than 2.45 at 25° cent. (77° Fahr.) and the percentage of absorption of water of the blocks, after being dried for 24 hours at a temperature of 65° cent. (149° Fahr.), should not be more than 1% after immersion in water for 7 days. The blocks should be about 5 in. in width and 12 in. in length, and 2, 2½, or 3 in. in depth, depending on traffic conditions.

Construction.—The blocks should be laid on a fresh ½-in. mortar bed which covers a cement-concrete foundation. After being laid, the blocks should be covered with a thin layer of clean, dry, fine, sharp sand which should be thoroughly swept into the joints until they are filled.

Bituminous Concrete Pavements.*

Classification.—The principles to be covered in drafting specifications for bituminous concrete pavements will be grouped under the

* For discussion of rates of grade, crown, artificial foundation, etc., see "General Conclusions."

three classes into which these pavements generally may be divided. These classes are described as follows:

Class *A*.—A bituminous concrete pavement having a mineral aggregate composed of one product of a crushing or screening plant;

Class *B*.—A bituminous concrete pavement having a mineral aggregate composed of a certain number of parts by weight or volume of one product of a crushing or screening plant and a certain number of parts by weight or volume of sand, broken stone screenings, or similar material, with or without a filler;

Class *C*.—A bituminous concrete pavement having a predetermined, mechanically graded aggregate composed of broken stone, broken slag, gravel, or shell, with or without sand, Portland cement, fine inert material, or combinations thereof.

Bituminous Concrete Pavements, Class *A*.

Mineral Aggregate.—Broken stone, because of the satisfactory bond secured, should be used wherever possible, although bituminous concretes constructed with gravel have proved satisfactory for light traffic where great care has been taken in the selection of the gravel and in the construction of the pavement.

Broken stone should be clean, rough-surfaced, sharp-angled, of compact texture, and uniform grain. If the pavement is to be subjected to medium or heavy traffic, the broken stone used for the construction of the wearing course should show a loss or abrasion of not more than 3.5%, and its toughness should not be less than 13.

Especial care is required in drafting the specifications covering the broken stone or gravel to be used. An excess of large or small sized stone or gravel should be avoided. Practice has demonstrated that a mineral aggregate composed of those materials which will comply with the following mechanical analysis, using laboratory screens having circular openings, will produce satisfactory results: All the material shall pass a 1¼-in. screen; not more than 10% nor less than 1% shall be retained on a 1-in. screen; not more than 10% nor less than 3% shall pass a ¼-in. screen.

Bituminous Cements.—Experience has demonstrated that the most efficacious bituminous concrete pavements of Class *A* are constructed by using suitable asphalt cements or refined tars in the mix, and asphalt cements for seal coats. Satisfactory results will be secured when tar cements are used for seal coats, but the surface must be re-treated more frequently than when asphalt cements are used.

Heating Aggregates and Bituminous Cements.—Although satisfactory pavements have been constructed using unheated mineral ag-

gregates and suitable bituminous cements, service tests demonstrate that the best results are secured by using for the mineral aggregate broken stone or gravel which is heated until thoroughly dry to between 66° cent. (150° Fahr.) and 121° cent. (250° Fahr.). If revolving dryers in which the flame is permitted to come in contact with the aggregate are used, great care should be taken to ensure uniformity of heating and so avoid the danger of burning the aggregate.

In order to obtain a fluidity of the bituminous material which should be sufficient to ensure a proper coating of the mineral particles in cases where a heated aggregate is used, and also to prevent injury to the bituminous material, the asphalt cements should be heated to a temperature between 135° cent. (275° Fahr.) and 177° cent. (350° Fahr.), and refined tars to a temperature between 93° cent. (200° Fahr.) and 135° cent. (275° Fahr.).

Mixing.—The quantity of bituminous cement to be used in the mix will depend on the kind of broken stone or gravel and bituminous cement, the character of the aggregate, the climatic conditions, etc. For the aggregate heretofore mentioned, the bituminous concrete mixture should contain between 5 and 8% by weight of bitumen.

The bituminous concrete should be mixed in mixers designed and operated so as to produce and discharge a thoroughly coated and uniform mixture of non-segregated aggregate and bituminous cement. Except on small contracts and for repair work, mixers which provide for the heating of the aggregate by the use of a flame in the mixing chamber should not be used, on account of the danger of burning the aggregate or the bituminous cement.

Laying.—To ensure ease of manipulation and the proper compaction of the bituminous concrete, the mixture as delivered on the roadway should have a temperature of not less than 66° cent. (150° Fahr.). Experience has demonstrated that a thickness, after rolling, of 2 in. of bituminous concrete is economical and efficacious. In order to secure an even surface and adequate compaction by a thorough interlocking of the particles of the aggregate, a tandem roller weighing between 10 and 12 tons should be used.

Seal Coat.—A seal coat should always be used on this type of bituminous concrete, as maintenance charges and annual cost will be reduced materially thereby. The seal coat should consist of from ½ to 1 gal. per sq. yd. of bituminous cement, uniformly distributed, preferably by the use of a hand-drawn distributor followed by a squeegee. The bituminous cement should be covered with an application of dry stone chips, which should be rolled.

Seasonal Limitations.—Bituminous concrete of this type should not be mixed or laid when the air temperature in the shade is lower

than 10° cent. (50° Fahr.), as otherwise it is difficult, under average conditions, to secure an even and well compacted wearing course.

Bituminous Concrete Pavements, Class *B*.

Specifications for pavements of this class have generally stipulated that so many parts of broken stone or gravel and so many parts of sand or other fine material are to be mixed with a certain quantity of bituminous cement. By the use of this specification, and with unusual supervision, it is practicable to secure a fairly well-graded aggregate, but in most cases the mixture will be found to contain an excess of broken stone, with insufficient fine material to fill the voids therein, and in other cases it will contain an excess of sand in which the broken stone is held as isolated particles.

Bituminous Concrete Pavements, Class *C*.

This type includes the so-called "Topeka" mixture, and several kinds of patented pavements.

Topeka Bituminous Concrete Pavement.—If the Topeka pavement specification embodies the grading, as contained in the decree of 1910, namely,

Bitumen, from 7 to 11 per cent.
Mineral aggregate, passing 200-mesh screen, from 5 to 11 per cent.
Mineral aggregate, passing 40-mesh screen, from 18 to 30 per cent.
Mineral aggregate, passing 10-mesh screen, from 25 to 35 per cent.
Mineral aggregate, passing 4-mesh screen, from 8 to 22 per cent.
Mineral aggregate, passing 2-mesh screen, less than 10 per cent.,

special provisions should be made in the specifications covering the broken stone and sand to be used, in order to secure satisfactory grading. Otherwise, the principles stated under "Bituminous Concrete Pavements, Class *A*", should be followed, except that a seal coat is not considered necessary under many conditions where this type of pavement is used.

Patented Bituminous Concrete Pavements.—In cases where patented bituminous concrete pavements of Class *B* or Class *C* are used, the same fundamental principles observed under "Bituminous Concrete Pavements, Class *A*" should be followed, especially in the case of covering in detail the composition and grading of the mineral aggregate, and the physical and chemical properties of the bituminous cements used.

Bituminous Macadam Pavements.*

Materials.—The broken stone should be of a quality equal to that prescribed for broken stone roads, and should have the same characteristics. The bituminous materials may be of asphalt or refined tar.

Construction.—The principles relating to thickness applicable to a broken stone road are likewise applicable to bituminous macadam pavements, and thorough rolling, including the rolling of the upper course, both before and after the application of the bituminous material, is also necessary. As it is desired to bind only the upper course with bituminous material, it is necessary, in order to prevent waste by penetration, that there should be no appreciable voids in the next lower course. It is not necessary, however, to flush the filler or binder in this course to the same extent as is necessary in binding the top course of a water-bound road, and it is absolutely essential that no binder should cover the stones of the lower course when the top course is spread.

The quantity of bituminous material used should be only sufficient to penetrate through the upper course. The quantity per square yard cannot be prescribed absolutely, depending in some degree on the hardness and size of stone used, but, in general, the application of 1 gal. or less to the square yard for each inch in thickness of the finished upper course is adequate.

The use of a pressure distributor in applying the bituminous material is essential, and the distributor should be of such type that absolutely uniform application may be accomplished, and that no ruts are formed in the surface by the wheels supporting the distributor.

The bituminous material should be applied at such a temperature that it will flow freely, and, to insure proper penetration, the stone should be dry and clean, and the air temperature should not be lower than 10° cent. (50° Fahr.) during application.

In order to secure a proper surface, the covering material should preferably consist of the crusher product passing over a $\frac{1}{4}$-in. screen and through a $\frac{3}{4}$-in. screen. Finer material, however, may be used for covering if a slippery surface is not objectionable, but the use of material passing through a 10-mesh sieve should be avoided.

The use of a bituminous material by no means justifies any lack of care in the ordinary details to be followed, but rather increases the need for thoroughness and skilled supervision. The main principles underlying good construction in water-bound roads remain in full force when such roads are treated with bituminous material. Whatever method may be used in any case, it is as essential, in bituminous work as in water-bound construction, that a suitable quality of road metal be used.

* For discussion of rates of grade, crown, artificial foundation, etc., see "General Conclusions."

Bituminous Surface Treatments.*

Description.—The proper treatment of a broken stone, gravel, shell, or slag roadway with bituminous material, for the purpose of eliminating the so-called dust nuisance, will at the same time render even the best of such roadways more efficient for sustaining traffic, and such treatment with bituminous materials is usually preferable and more economical than sprinkling with water, or the use of hygroscopic salts.

Bituminous Material.—Either refined tar, cut-back asphalt, or asphaltic oil may be used for surface treatment. If the surface to be treated is a gravel, broken stone, slag, or other porous and non-bituminous crust, practice has proven that bituminous material of such consistency that it can be applied at a temperature below 52° cent. (125° Fahr.) is preferable to heavier material, and that on any crust the application of a quantity in excess of ½ gal. per sq. yd. is inadvisable. It is advisable to apply the material in quantities not exceeding ¼ gal. per sq. yd. at a time. Heavier material in less quantity, however, should be used on bituminous roadways; otherwise there will be a tendency toward an objectionable softening of the material previously used in the construction of the roadway.

Construction.—The surface to which a bituminous treatment is to be applied should be dry, compact, and free from depressions and dust. On a broken stone road, the application should be made on the exposed stone surface of the upper course, such exposed surface being obtained by thoroughly removing with brooms or sweepers the binding material or dust that may have been applied or accumulated thereon. The bituminous material, in all cases, should be applied by a pressure distributor designed so that the material will be spread uniformly and with a pressure of not less than 20 or more than 75 lb. per sq. in.

The application, in all cases, should be carried over the outside edges of the rolled metal and on the shoulder far enough to protect the edges of the metaled surface. The material should be applied only after the surface has been thoroughly compacted by traffic or otherwise.

After the bituminous material is applied it should be covered with the toughest grit obtainable, preferably of a size that will pass through a screen having openings of not less than ⅜ in. nor greater than ⅝ in., just enough of such material being used to cover the bituminous material. It is advantageous, but not entirely necessary, to roll with a steam roller after the application of the grit.

Brick Pavements.†

Cushion Course.—In the case of brick pavements, the brick being of uniform size, sufficient resiliency will be secured by the use of a

* For discussion of rates of grade, crown, artificial foundation, etc., see "General Conclusions."

† For discussion of rates of grade, crown, artificial foundation, joints, etc., see "General Conclusions."

sand cushion 1 in., or even slightly less, in depth, provided that the depth is uniform and the surface of the concrete foundation is truly parallel with the finished pavement; and if the variation in the depth of the brick does not exceed $\frac{1}{8}$ in., the thickness of the sand cushion can safely be reduced to $\frac{3}{4}$ in. In many brick pavements recently laid the cushion course has been dispensed with entirely, the brick having been bedded in cement mortar spread over the concrete foundation. This results in a monolithic structure less capable of absorbing shock than is the case where a sand or bituminous cushion is introduced between the wearing surface and the foundation. A cushion course composed of sand or stone chips and a bituminous cement about $\frac{1}{2}$ in. in thickness may be substituted for sand or cement mortar, provided the surface of the concrete foundation is made sufficiently smooth and regular in contour.

Materials.—The quality of the brick should be determined by physical tests. The method of making the rattler test adopted by the American Society for Testing Materials is approved by the Committee. This test will indicate the toughness and resistance to wear from shock and abrasion. Uniformity in the rate of wear is so important that it properly may be a controlling consideration, even at the expense of a moderate increase in the rate of wear. In size and shape it is desirable to conform to accepted standards in order that repairs and renewals may more readily be made. Uniformity in size is especially important, and variations in depth should be kept within the narrowest limits.

Construction.—The bricks should be laid in straight courses at right angles to the axis of the roadway, although at intersections they may advantageously be laid in diagonal courses arranged so that traffic turning any of the corners will move across the bricks and not along the continuous joints. They should be laid so that the joints will be uniform in width and of sufficient width only to permit a filler to reach the bottom of the joints. Lug bricks have the advantage of insuring such uniform width with ordinary care in laying. If a sand cushion is used, great care should be taken to avoid any disturbance of the surface of the cushion after it has been brought to true grade by using a template. If bedded in a mortar or bituminous cushion, the brick should be bedded so that the surface will be as true as possible. In all cases the brick after being laid should be brought to a true and even surface by the use of a roller.

Broken Stone Roads.*

Materials.—All broken stone should be clean, rough-surfaced, angular, of compact texture, and of uniform grain. It should preferably be of such quality that, using standard laboratory tests, it will show a

* For discussion of rates of grade, crown, artificial foundation, etc., see "General Conclusions."

percentage of wear not greater than 5 (French coefficient of wear not less than 8), and a toughness of not less than 6.

The broken stone should be separated into component sizes by passing the product of the crusher over rotary screens having circular openings, the different sizes of stone being collected in separate bins. The separation of sizes is governed somewhat by the thickness at which the crust is to be laid. If a 6-in. surface is to be laid, the maximum size should not exceed that of stone passing a 3-in. screen, whereas, if the pavement is to be 7 in. or more in thickness, a 3½-in. screen is permissible. In all cases the stone should be spread and rolled in two or more courses, the largest size generally being used in the lower course and the smaller sizes in the upper course or courses, each course being spread in such manner that there will be uniformity in its density. Such uniformity is accomplished by spreading from piles dumped on boards or alongside the road, or by automatic spreading wagons.

The product passing through a ⅜ to ¾-in. screen should be used as a binder, and on two-course work should be applied on the top course only, whereas, on the three-course work, the second course should be lightly covered with this material.

Products of broken stone obtained from portable and stationary stone-crushing and screening plants, though complying with a given specification of the type now ordinarily used, vary to a considerable extent, due to variations in the plant and its operation, such as kind of rock, type and method of operation of crushing plants, methods of separation of products, differences in lengths and diameters of sections of rotary and shaker screens, and differences in inclination and rate of operation of screens.

The necessity for more carefully drawn specifications covering the sizes of the particles of which a given product of a stone-crushing and screening plant is composed, is illustrated by Table 3, mechanical analyses of two products obtained from the same plant, both of which products passed over a section of a rotary screen having circular holes 1¼ in., and through a section of a rotary screen having circular holes 2¼ in. in diameter.

TABLE 3.

	Sample "A." Percentage.	Sample "B." Percentage.
Passing ¼-in. screen	0.3	0.2
" ½ " " and retained on ¼-in. screen.	0.4	1.1
" ¾ " " " " " ½ " "	2.2	12.6
" 1 " " " " " ¾ " "	8.0	37.5
" 1¼ " " " " " 1 " "	29.1	40.9
" 1½ " " " " " 1¼ " "	27.1	7.7
" 2 " " " " " 1½ " "	32.9	0.0
	100.0	100.0

It is obvious that, for many forms of construction, in order to secure successful results, greater care must be used in the writing of specifications for products of broken stone. The Committee recommends the general use, as soon as practicable, of the "Standard Form of Specifications for Certain Commercial Grades of Broken Stone", as recommended by Committee D-4, of the American Society for Testing Materials, in its 1916 Report:

The broken stone shall consist of one product of the operation of a stone-crushing and screening plant, without re-combining or mixing, and shall conform to the following mechanical analysis, using laboratory screens:

Passing .... in. screen (having smallest holes selected) from .... to .... per cent.

Passing .... in. screen (having next to largest holes selected) from .... to .... per cent.

Passing .... in. screen (having largest holes selected) from .... to .... per cent.

In this form of specification an attempt is made to cover in the mechanical analysis only the limits of the smallest and largest particles. No attempt is made to secure a carefully graded aggregate, but simply a product suitable for the type of road or pavement in question.

An engineer should base the selection of screens, to be used in the specification for a given product of broken stone, on the results of mechanical analyses of many similar products obtained from portable and stationary crushing and screening plants which supply the locality in which the specification is to be used.

Construction.—Each course of a broken stone road should be thoroughly rolled with a roller weighing from 10 to 15 tons, the rolling being done first along the sides and gradually approaching the center, and being continued until there is no movement of the stone ahead of the wheels of the roller.

The binder should be used on the top course in such quantity that, after alternate spreading of binder and watering, with continuous rolling, the voids become so filled as to result in a wave of grout being pushed along the surface by the front wheel of the roller.

After the completion and binding of the top course, a thin layer of screenings or stone dust should be applied to the surface in sufficient quantity to cover it evenly.

Cement-Concrete Pavements.*

General.—A thickness of from 5 to 8 in., as stated in the general principles, may ordinarily be considered sufficient for a concrete slab, and, if it seems advisable, from motives of economy, the thickness may

* For discussion of rates of grade, crown, artificial foundation, joints, etc., see "General Conclusions."

be diminished from the center of the slab to the edges. Special conditions may call for variations, even outside of the limits given. The character and the drainage of the sub-grade, its probable stability as a foundation, as well as the nature and amount of traffic, are some of the factors which enter into the rational determination of the thickness of the slab.

Materials.—The cement should be tested by the methods recommended by the Special Committee on Uniform Tests for Cement* of the American Society of Civil Engineers, and should meet the requirements adopted by the American Society for Testing Materials, as printed in the 1916 Year Book of that Society.

The Committee wishes to emphasize the importance of the aggregate in making up the concrete structure. Fine aggregate may be considered as gravel, sand, or screenings from hard, durable rock, graded so that, when dry, it will pass a screen having ¼-in. circular openings. The best fine aggregate is in general that which is graded fairly uniformly from the ¼-in. size mentioned above, downward, but not more than 5% should be of such fineness that it will pass a sieve having 100 meshes per lin. in. A preponderance of the coarser particles rather than the finer is desirable, and, in any case, less than 3% of the material should pass the 200-mesh sieve. Standard briquettes, made of samples of fine aggregate and tested at the usual periods of 7 and 28 days, should show a strength at least equal to similar briquettes made of the same cement and three parts of standard Ottawa sand. Coarse aggregate, in general, should not be larger than will pass a screen with 2-in. circular openings, ranging down fairly uniformly from this size to that retained on a ¼-in. screen. The loss of such material, as determined by the abrasion test, should not be more than 5 per cent.

A denser and more uniform concrete may be made by screening the material, both fine and coarse aggregates, into different sizes and re-combining these different sizes in such a way as will give the densest mixture, that is, the smallest percentage of voids, when dry. Although this adds somewhat to the expense, the resulting composition will generally be found enough better to make up for the additional expenditure. Furthermore, in proportioning the ingredients of the concrete, a mixture based on mechanical analyses is in general to be preferred to the arbitrary rule of a 1:2:4 or 1:3:5 mix. The slight increase in time and expense which is incurred by determining the voids and combining the various sizes to get the greatest density is more than repaid by the strength and density of the resulting mixture. Similar care in the determination of the proper quantity of water is also to be desired in order that the concrete may have a uniform consistency as it is deposited in place. Though the mixture should be rather wet, especially if it is deposited without tamping, it should still be stiff

* *Transactions*, Am. Soc. C. E., Vol. LXXV, 1912, p. 665.

enough to hold its shape when struck off by the template, and yet not result in segregation of the different sizes throughout the mass. Water used in mixing should be clean and free from oil, alkali, or vegetable matter.

Construction.—Forms used in cement-concrete pavements should be as carefully considered as those for any other class of structural work in concrete. It is important they be true, and free from warp, and of sufficient strength to hold the wet concrete without springing out of shape. Particular care should be taken to keep the forms tight, so that leakage through the sides, which might allow the cement or mortar to be carried out of the coarse aggregate along the edges of the roadway, may be effectively prevented. The concrete should be deposited rapidly on the sub-grade to the required depth and to the entire width of the pavement. It is better to have the surface of the rolled and finished sub-grade thoroughly dampened before beginning to deposit the concrete. Rolling or ramming the freshly placed concrete is desirable wherever practicable, as it not only increases the density of the resulting mass, but also tends to place the particles of the coarse aggregate on the surface, so that they interlock with each other and present a flat side to the wear of traffic.

Clean vertical joints, straight across the roadway, through the entire mass of concrete in place, should be insisted on when the work stops for a day, or if there is a stoppage of more than 30 min. in the work during the day. Special precautions should be taken to prevent freezing when work is carried on during cold weather, and it should be borne in mind that concrete sets much more slowly in cold weather than in warm weather. If, in the course of the work, the temperature reaches, say, 40° Fahr., and is falling, the operation of mixing and laying concrete should be suspended, and the newly laid surface suitably protected from frost.

In finishing the surface of the concrete, a template or a striking board should be used, which gives the true form of the finished pavement for its entire width. For the final surface finish, the use of the wooden float, operated from a suitable bridge which spans the entire pavement, is desirable, although satisfactory results may be obtained by the use of a properly operated belt about 10 in. wide. Care should be taken that the final surface of the pavement is true, both transversely and longitudinally, that is, with regard to both cross-section and grade. In all cases the surface of the finished pavement should be kept wet and protected from the sun for several days.

Earth and Sand-Clay Roads.*

Materials for Earth Roads.—The earth road should be constructed of the natural soil, from which, for a depth of 10 in. at the center to

* For discussion of rates of grade, crown, artificial foundation, etc., see "General Conclusions."

5 in. at the sides of the roadway, all stones more than 3 in. in diameter and all sods, roots, and similar materials should be removed.

Construction of Earth Roads.—The soil should be mixed with plows, disk and spike-tooth harrows, or by other satisfactory means, until all the surfacing material is of uniform texture. When the earth roadway is completed its thickness should be not less than 10 in. at the center and 5 in. at the sides. The roadway should be shaped by the use of a road grader, and should be finished so as to conform to the desired cross-section, the crown of which should usually be about 1 in. to the ft. If any depressions appear during the grading of the roadway, they should be carefully filled with soil and the roadway reshaped. In order to secure satisfactory results, several months should elapse after the road is graded before it is considered complete; and such settlements and irregularities as develop should be corrected by the use of a grading machine or road drag.

Materials for Sand-Clay Roads.—Top-soil or mixtures of sand and clay should comply with the following requirements: The sand content should be at least from 70 to 80%; the sand used in sand-clay mixtures should preferably be composed of hard angular particles at least 30% of which should be retained on a 50-mesh sieve; the clay content should vary from 10 to 20% and under no circumstances should be allowed to exceed 30%; the clay, when placed on the roadway, should not contain lumps larger than 3 in. in diameter.

Construction of Natural Sand-Clay Roads.—The top-soil should be spread on a flat sub-grade to a depth of 10 or 12 in. at the center and 5 or 6 in. at the sides. After the surfacing material has been laid, the ditches should be constructed, and the material obtained therefrom should be used in constructing shoulders. It is advisable to plow and then harrow the surfacing material, as a more homogeneous layer is secured thereby. The surfacing material should not be compacted with an ordinary roller but with a sheep-foot roller, or the compaction should be obtained with the hoofs of animals and the wheels of vehicles going over the roadway. For successful results, thorough puddling is necessary, and this can practically only be secured through the medium of rains during the period of compaction. During compaction and after each rain the surface of the roadway should be immediately reshaped and crowned. When completed, the compacted roadway should have a depth of from 7 to 8 in. in the center and 3½ to 4 in. at the edges, and have a crown of about ½ in. per foot.

Construction of Sand-Clay Roads on Clay Sub-Soil.—Although there are several methods of construction, the following is advocated, as uniformly satisfactory results are secured by its use. The roadbed should be graded practically level, and the portions to be constructed of sand-clay should be excavated to a sufficient depth so that the earth

removed and placed on the shoulder of the road will give the right slope to the ditches. The depth of excavation should be from $1\frac{1}{2}$ to 3 in., depending on the width of the road between ditches. The clay soil should then be plowed to a depth of 2 or 3 in., after which about 4 in. of sand should be spread evenly over the surface and thoroughly worked in with a disk harrow. Then 4 in. more of sand should be added and harrowed. During a rain, the sand-clay mixture should be thoroughly harrowed, and after the rain the roadway should be dragged into shape.

Construction of Sand-Clay Roads on Sand Sub-soil.—On a practically flat roadbed, a layer of clay from 2 to 4 in. in thickness should be spread evenly over the roadway. A layer of clean sand should then be spread over the clay, and the roadway thoroughly harrowed. The roadway should be shaped up with a drag, and during a rain it should be again harrowed. The roadway should be finally shaped with the use of a grading machine or drag.

Gravel Roads.*

Description.—The subject of gravel roads embraces a great variety of styles of construction, from the simple expedient of surfacing the existing roadway with run-of-the-bank gravel from near-by pits to the construction of an improved highway with all the necessary drainage structures and improvements of grade attending a broken stone roadway.

Materials and Construction.—The method of treating the gravel itself may vary from the application as it is found in the pit to the separation into sizes, and even to passing the gravel through the crusher before screening. In general, we may separate gravel roadways into two classes:

First, those in which the gravel is screened and applied in the same manner as with a broken stone roadway. This may be with or without crushing the gravel.

Second, those in which the gravel is applied to the roadway in its natural state as found in the pit, with or without the addition of other material, or the natural material may be passed through the crusher before application to the roadway.

In the first case, with rounded gravel, the tendency toward dislodgment under traffic is greater than with angular broken stone, and hence, in order to reduce this tendency, the size of the pieces in the courses of the roadway must be somewhat smaller with gravel than in the case of broken stone.

In the second case, the selection of the material will be governed by the gravel available in the locality where the roadway is to be constructed. Every endeavor should be made to select a material that

* For discussion of rates of grade, crown, artificial foundation, etc., see "General Conclusions."

will show, by test, its fitness as regards hardness, toughness, and cementing power. The gravel should show, on mechanical analysis, a grading of material which will contain sufficient stone of the larger size to insure stability and wearing qualities under traffic, and which will contain sufficient finer material to insure a proper bond. The best material will be composed of gravel graded so that it will have a maximum density.

Though the most available material will have a wide range as to sizes, the Committee believes that the following specification for sizes, adopted by the American Society of Municipal Improvements in 1916, may be followed safely:

"Two mixtures of gravel, sand, and clay shall be used, hereinafter designated in these specifications as No. 1 product (for top course) and No. 2 product (for middle and bottom courses).

"No. 1 product shall consist of a mixture of gravel, sand, and clay, with the proportions of the various sizes as follows: All to pass a 1½-in. screen and to have at least 60 and not more than 75 per cent. retained on a ¼-in. screen; at least 25 and not more than 75 per cent. of the total coarse aggregate (material over ¼ in. in size) to be retained on a ¾-in. screen; at least 65 and not more than 85 per cent. of the total fine aggregate (material under ¼ in. in size) to be retained on a 200-mesh sieve.

"No. 2 product shall consist of a mixture of gravel, sand, and clay, with the proportions of the various sizes as follows: All to pass a 2½-in. screen and to have at least 60 and not more than 75 per cent. retained on a ¼-in. screen; at least 25 and not more than 75 per cent. of the total coarse aggregate to be retained on a 1-in. screen; at least 65 and not more than 85 per cent. of the total fine aggregate to be retained on a 200-mesh sieve."

With gravel such a quartz, the cementation of which is low, a highly cementitious void filler is desirable, and a moderate quantity of clay or loam may be permissible.

A more generous use of gravel, especially in the surfacing of earth roads, should be encouraged. The low first cost and ease of maintenance should help materially to increase the mileage of serviceable roads in the country districts.

In drafting specifications, the refinements to be used in the methods of construction will depend on the kind and amount of traffic to be sustained, the character and quality of gravel to be secured in that particular locality, and other local conditions. Generally, however, the refinements should be carried to the point where further refinement would not be economical.

SHEET-ASPHALT PAVEMENTS.*

General.—A sheet-asphalt wearing course, consisting of predetermined graded sand, filler, and asphalt cement, should be laid to a

* For discussion of rates of grade, crown, artificial foundation, etc., see "General Conclusions."

compacted thickness of not less than 1½ in. and not more than 2 in., on a binder course of bituminous concrete consisting of broken stone or broken stone and sand mixed with asphalt cement, the binder course having a compacted thickness of not less than 1 in. nor more than 1½ in.

Materials.—For heavy or medium traffic, the so-called close binder should be used, instead of the open binder, as the former possesses greater inherent stability than the latter. Specifications for the grading of open binder should be similar to those for the aggregate for Bituminous Concrete Pavements, Class *A*; and, for a close binder, similar to the following:

Ninety-five per cent. of the binder aggregate shall pass a screen having circular openings the diameter of which shall be of three-quarters the thickness of the binder course to be laid. The remaining 5% shall not exceed in their smallest dimension the thickness of the binder course to be laid. The binder aggregate shall be graded from coarse to fine, so as to have the following mesh composition:

Passing 10-mesh sieve..........	15 to 35%	Total passing ½-in. screen, 35 to 85%.
Passing ½-in. screen and retained on 10-mesh sieve...........	20 to 50%	

The sand for the wearing course shall be carefully graded. For pavements to be subjected to medium or heavy traffic, there should be a preponderance of the finer particles; and for pavements to be subjected to light traffic, there may be a preponderance of the coarser particles. Specifications for a sand for wearing courses to be subjected to medium or heavy traffic should be similar to the following: The sand shall be hard, clean, and moderately sharp. On sifting it shall have the following mesh composition:

Passing 200-mesh	0 to 5%	
" 100-mesh and retained on 200-mesh	10 " 25%	Total passing 80-mesh and retained on 200-mesh, 20 to 40%.
" 80 " " " " 100 "	6 " 20%	
" 50 " " " " 80 "	5 " 40%	
" 40 " " " " 50 "	5 " 30%	
" 30 " " " " 40 "	5 " 25%	Total passing 10-mesh and retained on 40-mesh, 12 to 45%.
" 20 " " " " 30 "	5 " 15%	
" 10 " " " " 20 "	2 " 15%	

The filler should be thoroughly dry limestone dust, or dust from other equally satisfactory stone, or Portland cement, the whole of which should pass a 30-mesh sieve and at least 66% of which should pass a 200-mesh sieve. The surface mixture should contain from 6 to 20% of this filler, depending on the kind of sand and asphalt used and the traffic conditions on the street or streets to be paved.

The specifications should contain detailed requirements covering the physical properties of the asphalt cement, and should prescribe the bitumen content of the binder course and sheet-asphalt wearing course mixture. For the grading mentioned, the bitumen should be, for the close binder, from 4 to 7%; and for the sheet-asphalt wearing course mixture, from 9.5 to 13.5 per cent.

Construction.—The broken stone for the binder should be heated to a temperature between 107° cent. (225° Fahr.) and 177° cent. (350° Fahr.). The sand when mixed with the asphalt cement should have a temperature between 135° cent. (275° Fahr.) and 190° cent. (375° Fahr.). The asphalt cement when used should have a temperature between 121° cent. (250° Fahr.) and 177° cent. (350° Fahr.).

The asphalt cement and broken stone, or broken stone and sand, for the binder course, and the asphalt cement, sand, and filler for the wearing course, should be thoroughly mixed by machinery until a uniform mixture is produced in which all the particles are thoroughly coated with asphalt cement.

When brought to the work, the temperature of the binder mixture should be between 93° cent. (200° Fahr.) and 163° cent. (325° Fahr.), and of the wearing course mixture between 110° cent. (230° Fahr.) and 177° cent. (350° Fahr.). The binder course and the wearing surface should be compacted separately by rolling with a self-propelled roller weighing not less than 200 lb. per inch of width of tread, the rolling being carried on continuously at the rate of not more than 200 sq. yd. per hour per roller until a satisfactory compression is obtained. Excessive use of water on the steam roller while compacting the courses of the pavement should not be permitted. During the rolling of the wearing course, a small quantity of Portland cement should be swept over its surface. In cases where sheet-asphalt is constructed next to the curb, it is advisable to coat the surface for a space of 12 in. next to the curb with hot asphalt cement.

Stone Block Pavements.*

Materials.—The stone pavements of this country are generally of granite or sandstone, the particular kind being determined by the availability of the different materials. Limestone is used to a certain extent in one or two cities, but so slightly that it need not be considered.

In order to make a good paving block, stone should be resistant to wear, hard and tough, and of such a character as to be easily broken into regular shapes. Toughness is more important than hardness, as in very few cases will the blocks of a stone pavement, of a character such as is generally used, be much reduced under actual traffic. The quantity of wear is not as important as that the wear shall be uniform, so that the surface of the pavement may be kept smooth and even.

* For discussion of rates of grade, crown, artificial foundation, joints, etc., see "General Conclusions."

The character of sandstone is such that, though it does wear smooth, it is never slippery; but with granite, if the stone is too hard, even when no particular wear is noticed under traffic, the surface soon becomes smooth and slippery, and the harder the stone the more slippery it becomes.

To make suitable paving blocks, granite should be a medium and uniform-grained stone, of such a character that when broken it will present smooth and even surfaces, and have a percentage of wear of not more than 4.5 and toughness of not less than 8. It should have a crushing strength of not less than 20 000 lb. per sq. in.

Sandstone should be hard and tough, and of a character to meet the requirements given for granite, except as to crushing strength, which should be not less than 16 000 lb. per sq. in.

As the stone must be made into comparatively small blocks, and as this is done by expensive labor, the size of the blocks is extremely important. Probably the ideal sized block would be 8 in. long, $3\frac{1}{2}$ in. wide, and, under general conditions, 5 in. deep, but if blocks were made to conform exactly to these dimensions, they would be extremely expensive, so that it is considered good practice to allow variations in length from 8 to 12 in., in width from $3\frac{1}{2}$ to $4\frac{1}{2}$ in., and in depth from $4\frac{3}{4}$ to $5\frac{1}{2}$ in., and on light-traffic streets, where certain conditions make a stone pavement desirable, an even less depth may be permitted. When sandstone is used, the blocks may be a little wider, as far as use is concerned. All blocks, however, should be sorted, so that the adjacent courses can be kept as nearly uniform in width as possible. Under no circumstances, however, should blocks of different widths be used in the same course. The blocks should be dressed so that they will lie with close joints, and have good, smooth, and even heads.

Another form of stone block pavement, used to some extent in Europe, and which has recently been introduced into this country, is known as "Durax" in England and as "Kleinpflaster" in Germany. It consists of blocks approximating cubes $2\frac{1}{2}$ to 4 in. in size, although they should not be exactly cubical; they should be sufficiently irregular, both in size and shape, to permit them to be laid in arcs of circles of comparatively small radii and so that the joints will not be excessively large. By laying the courses in circular arcs, none of the joints is parallel to any line of traffic.

In Europe these blocks are used to a great extent in resurfacing the broken stone roads where the traffic is too heavy for macadam, and to some extent in city streets. If the blocks can be produced in this country at a reasonable price, they will make very satisfactory roads. They have been used to a slight extent in pavements in some Southern cities.

During the last few years many pavements have been laid with granite blocks made by splitting up old ones which had been in use

for some years. With blocks that ranged from 4 to 5 in. in width, 10 to 12 in., and even 14 in., in length, and 8 in. in depth, it has been found possible to get many good blocks of smaller size by cutting them up. The new blocks, being small, could be cut to a reasonably true surface without much work, with the result that the old blocks recut would actually lay more square yards in a pavement than the original ones. This practice is to be commended, both on the score of economy and result.

Foundation and Cushion.—It is assumed that the foundation for permanent stone pavements will in all cases be concrete. On the concrete must be spread a material to act both as a cushion to the blocks themselves and to even up the surface of the concrete; and the smoother the surface and the less the variation in the depth of the blocks, the thinner can be the cushion, although it should not be less than ¾ in., in any event.

The cushion which has generally been used for this purpose is sand, but recently engineers have been considering the advisability of using Portland cement mortar instead. The objection to mortar, made by some, is that it makes a too solid base for the blocks, not giving any resiliency. This at present is a mooted question, and the Committee does not desire to express a positive opinion as to the relative values of the two. If a good bituminous cushion could be provided, it would probably be more satisfactory than either the mortar or the sand, but it is questionable if the advantage gained would justify the increase in expense. (See also Brick Pavements.)

Construction.—The blocks should be laid on the cushion stone to stone, keeping the joints as small as possible. The joints should be filled with water-proof material. For this purpose a Portland cement grout, asphalt, or some other bituminous filler is generally used, and in some cases sand is mixed with a bituminous material in order to increase the toughness of the filler. All these fillers give good results, but with cement grout the cost of taking up and restoring the pavement over cuts is increased over that of a bituminous filler, as in many cases blocks are broken in taking them up, and it is difficult to clean the cement from the individual blocks and also to keep the traffic from the cut or patch while the grout is setting after the pavement has been restored. Another disadvantage of the cement grout filler is that it is highly important that it be perfectly set before traffic is allowed on the pavement, and in large cities it is almost impossible to keep traffic from the pavement, after it is laid, for the necessary length of time.

Wood Block Pavements.*

General.—In using wood in pavements, special attention should be given to the crown of the street, as this material undoubtedly presents,

* For discussion of rates of grade, crown, artificial foundation, etc., see "General Conclusions."

under certain conditions, a more slippery surface to traffic than any other. Wherever the longitudinal grade is sufficient to allow the water to run off freely, the crown should be very flat, not exceeding 3 in. in a roadway width of 30 ft. On streets which must be used continuously, the maximum grade allowed should not exceed 2%, although on residence streets, where pavements can be avoided when they are exceptionally slippery, grades up to 3 or 4% are permissible. In the Middle West, where, as a rule, there is less moisture in the air under ordinary conditions, the foregoing grades have been exceeded with satisfactory results.

Kinds of Wood for Blocks.—Whatever the kind of wood used for pavements, it must be treated with some preservative, in order to make it suitable. It is important that as many kinds of wood be utilized as possible, so that in any territory the most available one can be used. Just how many varieties can be utilized is uncertain at the present time, but those that are undoubtedly good are: Southern yellow pine, Douglas fir, tamarack, Norway pine, hemlock, and black gum. In the East and Central West, Southern yellow pine, and on the Pacific Coast, Douglas fir, are generally used. Experiments will be necessary to determine just what other kinds will be satisfactory.

The blocks must be sound, and must be well manufactured, square-butted, square-edged, free from unsound, loose or hollow knots, knot holes, worm holes, and other defects, such as shakes, checks, etc., that would be detrimental to the blocks.

The number of annual rings in the 1 in. which begins 2 in. from the pith of the block should not be less than 6, measured radially, provided, however, that blocks containing between 5 and 6 rings in this inch may be accepted if they contain 33⅓%, or more, of summer wood. In case the block does not contain the pith, the 1 in. to be used shall begin 1 in. away from the ring which is nearest to the heart of the block. The blocks in each charge shall contain an average of at least 70% of heart wood. No one block shall be accepted that contains less than 50% of heart wood.

Size of Blocks.—The blocks should be from 5 to 10 in. long, but should preferably average two times the depth. The Committee recommends blocks 4 in. in depth for very heavy traffic streets; blocks 3½ in. in depth for moderate traffic streets. For light traffic streets, 3 in. in depth may be used, but where 3-in. blocks are used, no blocks should be longer than 8 in. They may be from 3 to 4 in. in width, but, in any one city block, all of them should be of uniform width. A variation of $\frac{1}{16}$ in. should be allowed in the depth and ⅛ in. in width of the blocks from that specified. In all cases the width should be greater or less than the depth by at least ½ in.

Preservatives for Wood Blocks.—Many different materials have been used in the past for wood preservatives, but the Committee believes

that, taking all things into consideration, coal-tar creosote oil is the best. Some engineers, however, feel that a creosote oil produced from water-gas tar is as good as one produced from coal-gas tar, if not better. As the object of the preservative is not only to prevent the blocks from decay, but also to prevent them from swelling in wet weather or shrinking in dry weather, whatever the preservative, it should be of a character that will render the blocks stable and free from decay, for as long a time as possible. It is probable that under the traffic that prevails on most of the streets paved with wood in this country, if the blocks can be kept stable and free from decay, the pavement will last from 30 to 35 years, or even longer. It is necessary, however, to have an oil that is in itself stable and will remain in the blocks a long time.

It is thought that a heavy gravity oil will do this better than one of light gravity, as the former is less volatile and will maintain its condition better while exposed to atmospheric changes. The Committee recognizes that good results have been obtained by the use of a pure distillate oil, and also one which contains a certain quantity of coal-gas tar.

Treatment of Wood Blocks.—The timber may be either air-seasoned or green, but should preferably be treated within 3 months from the time it is sawed. Green timber and seasoned timber, however, should not be treated together in the same charge.

In any charge, blocks should contain at least 16 lb. of water-free oil per cubic foot of wood at the completion of the treatment. The blocks after treatment should show satisfactory penetration of the preservative, and in all cases the oil must be diffused throughout the sapwood. To determine this, at least 25 blocks shall be selected from various parts of each charge and sawed in half perpendicular to the fibers through the center, and if more than one of these blocks shows untreated sapwood, the charge should be re-treated. After re-treating, the charge should be again subjected to a similar inspection.

The surface of the blocks after treatment should be free from deposits of objectionable substances, and all blocks that have been materially warped, checked, or otherwise injured in the process of treatment should be rejected.

Handling Blocks After Treatment.—Blocks should preferably be laid in the street as soon as possible after being treated. If they cannot be laid immediately, provision should be made to prevent them from drying out by stacking in close piles and covering them, and, if possible, by sprinkling them thoroughly at intervals. In any case, where they are not laid as soon as they are received on the street, they should be well sprinkled about 2 days before being laid, under the direction of the purchaser. It is important to have the wood sufficiently wet to be swelled to its maximum size before it is laid.

Inspection.—All material specified and processes used in the manufacture of the blocks therefrom should be subject to inspection, acceptance, or rejection at the plant of the manufacturer, which should be equipped with the necessary gauges, appliances, and facilities to enable the inspector to satisfy himself that the requirements of the specifications are fulfilled.

The purchaser should have the further right to inspect the blocks after delivery on the street, for the purpose of rejecting those that do not meet these specifications, except that the plant inspections should be final with respect to the kind of wood, rings per inch, oil, and treatment.

Construction.—There are two methods of laying wood blocks, one with and one without a cushion. In Europe it is invariably the practice to lay the blocks directly on the concrete bed. When this is done it is necessary that the surface of the concrete be made absolutely smooth and true to the required cross-section of the pavement. In this country the practice has been to surface-up the concrete, as has been mentioned in connection with stone blocks, with cement mortar or sand. The Committee believes that the cement mortar will give a better result than the sand. It should, however, be mixed as dry as possible and at the same time insure setting, and the blocks should be thoroughly rolled into it. If the sand cushion is used, the blocks should also be rolled to a smooth surface.

The Committee looks with a great deal of favor on the practice of finishing the concrete to a true surface and laying the blocks directly on it, and would suggest that engineers laying this pavement try this method, and, if the cost of producing a smooth concrete surface is not excessive, that the method be generally adopted.

The blocks should be laid closely and the joints filled with some suitable material. Three materials have been used in this country for joint filling: sand, cement grout, and a bituminous material. On heavy-traffic streets, if fine sand is used, good results will be obtained. On light-traffic streets, however, it may be better to use a bituminous filler of practically the same character as that used for granite. If a bituminous filler is used, the pavement should be covered with a thin layer of fine sand, which should be allowed to remain 1 or 2 weeks after the traffic has been allowed on the street. The Committee does not feel that cement grout should be used, in any case. Though a wood block pavement should keep stable, it is undoubtedly safer to use a bituminous joint along the curb to provide for expansion or contraction.

Bleeding.—Considerable inconvenience has been caused in certain cities by "bleeding", or the exudation of the preservative on the surface of the street, after the blocks have been laid. If the proper precaution is taken with the character of the material and the character

of the treatment, it is thought that this can be avoided. If the pavement, however, should bleed to such an extent as to be a nuisance, it should be covered with fine sand, so that the surface material can be absorbed. After one or two applications there should be no further trouble.

DEFINITIONS.

For the sake of uniformity and clearness, the Committee recommends that the following lists of terms of frequent use in expressions relating to highway work be recognized as having the meanings set forth in the list, unless otherwise definitely stated at any time by a user of such term or terms.

In the list given will be found some terms and definitions adopted by the American Society for Testing Materials noted thus *, others by the American Reporters on Communication No. 10 at the Third International Road Congress, designated thus †, and other terms and definitions which have been proposed by the Committee on "Standard Tests for Road Materials" (Committee D-4) of the American Society for Testing Materials, which have been indicated thus ‡. The Committee wishes to acknowledge here its obligations for suggestions thus reaching it.

Aggregate.—The inert material, such as sand, gravel, shell, slag, or broken stone, or combinations thereof, with which the cementing material is mixed to form a mortar or concrete.

Asphalt.—† Solid or semi-solid native bitumens, solid or semi-solid bitumens obtained by refining petroleums, or solid or semi-solid bitumens which are combinations of the bitumens mentioned with petroleums or derivatives thereof, which melt on the application of heat, and which consist of a mixture of hydrocarbons and their derivatives of complex structure, largely cyclic and bridge compounds.

Asphalt Block Pavement.—One having a wearing course of previously prepared blocks of asphaltic concrete.

Asphalt Cement.—A fluxed or unfluxed asphaltic material, especially prepared as to quality and consistency, suitable for direct use in the manufacture of asphaltic pavements, and having a penetration of between 5 and 250.

Asphaltenes.—† The components of the bitumen in petroleum, petroleum products, malthas, asphalt cements, and solid native bitumens, which are soluble in carbon disulphide, but insoluble in paraffin naphthas.

Asphaltic.—Similar to, or essentially composed of, asphalt.

Base.—Artificial foundation.

Binder.—(1) A foreign or fine material introduced into the mineral portion of the wearing surface for the purpose of assisting the road metal to retain its integrity under stress, as well as, perhaps,

to aid in its first construction. (2) The course, in a sheet-asphalt pavement, frequently used between the concrete foundation and the sheet-asphalt mixture of graded sand and asphalt cement.

Bitumen.—* A mixture of native or pyrogenous hydrocarbons and their non-metallic derivatives, which may be gases, liquids, viscous liquids, or solids, and which are soluble in carbon disulphide.

Bituminous Cement.—A bituminous material suitable for use as a binder having cementing qualities which are dependent mainly on its bituminous character.

Bituminous Concrete Pavement.—One composed of broken stone, broken slag, gravel, or shell, with or without sand, Portland cement, fine inert material, or combinations thereof, and a bituminous cement incorporated together by a mixing method.

Bituminous Macadam Pavement.—One having a wearing course of macadam with the interstices filled by a penetration method with a bituminous binder.

Bituminous Material.—Material containing bitumen as an essential constituent.

> *Liquid Bituminous Material.*—‡ Bituminous material showing a penetration at normal temperature under a load of 50 grammes applied for 1 sec. of more than 350.
>
> *Semi-Solid Bituminous Material.*—‡ Bituminous material showing a penetration at normal temperature under a load of 100 grammes applied for 5 sec. of more than 10, and under a load of 50 grammes applied for 1 sec. of not more than 350.
>
> *Solid Bituminous Material.*—‡ Bituminous material showing a penetration at normal temperature under a load of 100 grammes applied for 5 sec. of not more than 10.

Bituminous Pavement.—One composed of broken stone, broken slag, gravel, shell, sand, or fine inert material, or combinations thereof, and bituminous cement incorporated together.

Bituminous Surface.—A superficial coat of bituminous material with or without the addition of stone or slag chips, gravel, sand, or material of similar character.

Blanket.—See "Carpet."

Bleeding.—The exudation of bituminous material on the roadway surface after construction.

Blown Petroleums.—† Semi-solid or solid products produced primarily by the action of air upon originally fluid native bitumens which are heated during the blowing process.

Bond.—The combined action of inertia, friction, and of the forces of adhesion and cohesion which helps the separate particles composing a crust or pavement to resist separation under stress. Mechanical bond is the bond produced almost wholly, in a well-built broken-stone

macadam road, by the interlocking of angular fragments of stone and the subsequent filling of the remaining interstices with the finer particles.

Bound.—Bonded.

Water-Bound.—Bonded with the aid of water.

Bituminous Bound.—Bonded with the aid of bituminous material.

Brick Pavement.—One having a wearing course of paving bricks or blocks.

Bridge.—A structure for the purpose of carrying traffic over a gap in the roadbed measuring 10 ft. or more in the clear span.

Camber of a Bridge.—The rise of its center above a straight line through its ends.

Camber of a Road.—See "Crown."

Carbenes.—† The components of the bitumen in petroleums, petroleum products, malthas, asphalt cements, and solid native bitumens, which are soluble in carbon disulphide, but insoluble in carbon tetrachloride.

Carpet.—A bituminous surface of appreciable thickness, generally formed on top of a roadway by the application of one or more coats of bituminous material with gravel, sand, or stone chips added.

Cement.—An adhesive substance used for uniting particles of other materials to each other. Ordinarily applied only to calcined "cement rock", or to artificially prepared, calcined, and ground mixtures of limestone and silicious materials. Sometimes used to designate bituminous binder used in bituminous pavements, when the expression "bituminous cement" (q. v.) is understood to be meant.

Cement-Concrete.—An intimate mixture of gravel, shell, slag, or broken stone particles with certain proportions of sand or similar material, cement, and water, made previous to placing.

Cement-Concrete Pavement.—One having a wearing course of hydraulic cement concrete.

Cemented.—Bonded. Referring to water-bound macadam, the term "cemented" is used to designate that condition existing when, after rolling the stone forming the crust, the remaining voids have been filled with the finer sizes, and the stone dust or "flour" has, under the action of water, taken a "set", as does cement itself.

Chips.—Small angular fragments of stone or slag containing no dust.

Clay.—Finely divided earth, generally silicious and aluminous, which will pass a 200-mesh sieve. Also see "Gravel."

Coal-Tar.—† The mixture of hydrocarbon distillates, mostly unsaturated ring compounds, produced in the destructive distillation of coal.

Coat.—See "Carpet." (1) The total result of one or more single surface applications. (2) To apply a coat.

Coke-Oven Tar.—† Coal-tar produced in by-product coke ovens in the manufacture of coke from bituminous coal.

Consistency.—* The degree of solidity or fluidity of bituminous materials.

Course.—One or more layers of road metal spread and compacted separately for the formation of the road or pavement. Courses are usually referred to in the order of their laying as first course, second course, third course, etc. Also a single row of blocks in a pavement.

Crown.—The rise in cross-section from the lowest to the highest part of the finished roadway. It may be expressed either as so many inches (or tenths of a foot), or as a rate per foot of distance from side to center, *i. e.,* "the crown is 4 in.", or "the crown is ½ in. to the foot."

Crusher Run.—The total unscreened product of a stone crusher.

Crusher-Run Stone.—The product of a stone-crusher, unscreened except for the removal of the particles smaller than remaining on about a ¼-in. screen.

Crust.—That portion of a macadam or similar roadway above the foundation consisting of the road metal proper with its bonding agent or binder.

Culvert.—A structure for the purpose of carrying traffic over a gap in the road-bed, measuring less than 10 ft. in clear span.

Cut-Back Products.—Petroleum, or tar residua, which have been fluxed, each with its own or similar distillates.

Dead Oils.—* Oils, with a density greater than water, which are distilled from tars.

Dehydrated Tars.—† Tars from which all water has been removed.

Ditch.—The open-side drain of a roadway, usually deep in proportion to its width, and unpaved.

Drainage.—Provision for the disposition of water.

> *Side-Drainage.*—That along the sides of the roadway.
>
> *Sub- or Under-Drainage.*—That below the surface.
>
> *Surface Drainage.*—That on the roadway or ground surface.
>
> *V-Drainage.*—That provided by the construction of troughs in the sub-grade of the roadway which troughs are like a "V", with flat sloping sides, and are filled with stone.

Dust.—Earth or other matter in fine, dry particles, so attenuated that they can be raised and carried by air currents. The product of the crusher passing through a fine sieve.

Dust Layer.—Material applied to a roadway for temporarily preventing the formation or dispersion under traffic of distributable dust.

Earth Road.—A roadway composed of natural earthy material.

Emulsion.—A combination of water and oily material made miscible with water through the action of a saponifying or other agent.

Expansion Joint.—A separation of the mass of a structure, usually in the form of a joint filled with elastic material, which will provide opportunity for slight movement in the structure.

Fat.—Containing an excess. A fat asphalt mixture is one in which the asphalt cement is in excess and the excess is clearly apparent.

Filler.—(1) Relatively fine material used to fill the voids in the aggregate. (2) Material used to fill the joints in a brick or block pavement.

Fixed Carbon.—* The organic matter of the residual coke obtained upon burning hydrocarbon products in a covered vessel in the absence of free oxygen.

Flour.—† Finely ground rocks or minerals pulverized to an impalpable product.

Flush Coat.—See "Seal Coat."

Flushing.—(1) Completely filling the voids. (2) Washing a pavement with an excess of water.

Flux.—† Bitumens, generally liquid, used in combination with harder bitumens for the purpose of softening the latter.

Footway.—The portion of the highway devoted especially to pedestrians. A sidewalk.

Foundation.—The portion of the roadway below and supporting the crust or pavement.

> *Artificial Foundation.*—That layer of the foundation especially placed on the sub-grade for the purpose of reinforcing the supporting power of the latter itself, and composed of material different from that of the sub-grade proper.

Free Carbon.—* In tars, organic matter which is insoluble in carbon disulphide.

Gas-House Coal-Tar.—† Coal-tar produced in gas-house retorts in the manufacture of illuminating gas from bituminous coal.

Grade.—(1) The profile of the center of the roadway, or its rate of rise or fall. (2) Elevation. (3) To establish a profile by cuts and fills or earthwork. (4) To arrange by sizes, broken stone, gravel, sand, or combinations of such materials.

Gravel.—Small stones or pebbles usually found in natural deposits more or less intermixed with sand, clay, etc., but in which mixture the particles which will not pass a 10-mesh sieve predominate.

> *Pea Gravel.*—Clean gravel the particles of which approximate peas in size.

Grit.—Stone chips, slag chips, or small gravel.

Gutter.—The artificially surfaced and generally shallow waterway provided usually at the sides of the roadway for carrying surface drainage. Occasionally used synonymously with "ditch", but incorrectly so, as "gutters" are always paved or otherwise surfaced, and ditches are not.

Haunches.—The sides or flanks of a roadway. Sometimes also called "quarters."

Highway.—The entire right of way devoted to public travel, including the sidewalks and other public spaces, if such exist.

Layer.—A course made in one application.

Loam.—Finely divided earthy material containing a considerable proportion of organic matter.

Macadam.—A road crust composed of stone or similar material broken into irregular angular fragments compacted together so as to be interlocked and mechanically bound to the utmost possible extent.

Mastic.—A mixture of bituminous material and fine mineral matter suitably made for use in highway construction and for application in a heated condition.

Mat.—† See "Carpet."

Matrix.—* The binding material or mixture of binding material and fine aggregate in which the large aggregate is embedded or held in place.

Mesh.—The square opening of a sieve.

Metal.—See "Road-Metal."

Mortar.—A mixture of fine material such as sand, cement, and water or other liquid suitably proportioned and incorporated together for the purpose for which it is used.

Normal Temperature.—‡ As applied to laboratory observations of the physical characteristics of bituminous materials, is 25° cent. (77° Fahr.).

Oil-Gas Tars.—* Tars produced by cracking oil vapors at high temperatures in the manufacture of oil gas.

Palliative.—A short-lived dust layer.

Patching.—Repairing or restoring small isolated areas in the surface of the metaled or paved portion of the highway.

Pavement.—The wearing course of the roadway or footway, when constructed with a cement or bituminous binder, or composed of blocks or slabs, together with any cushion or "binder" course.

Penetration.—In laboratory investigations, the distance, expressed in hundredths of a centimeter, entered a sample by a No. 2 cambric needle, operated in a machine for the purpose, and under known conditions of loading, time, and temperature. Where the conditions of test are not specifically mentioned, the load, time, and temperature are understood to be 100 grammes, 5 sec., and 25° cent. (77° Fahr.), re-

spectively, and the units of penetration to indicate hundredths of a centimeter.

Penetration Method.—The method of constructing a bituminous-macadam pavement by pouring or grouting the bituminous material into the upper course of the road metal before the binding of the latter has been completed.

Pitch.—† Solid residue produced in the evaporation or distillation of bitumens, the term being usually applied to residue obtained from tar.

Hard Pitch.—Pitch showing a penetration of not more than ten.

Soft Pitch.—Pitch showing a penetration of more than ten.

Straight-Run Pitch.—‡ A pitch run in the initial process of distillation, to the consistency desired without subsequent fluxing.

Pocket.—A hole or depression in the wearing course.

Pot-Hole.—A hole extending below the wearing course.

Profile.—A longitudinal section of a highway, generally taken along the center line.

Quarters.—The four sections of equal width which, side by side, make up the total width of a roadway.

Raveling.—The loosening of the metal composing the crust.

Refined Tar.—† A tar freed from water by evaporation or distillation which is continued until the residue is of desired consistency or a product produced by fluxing tar residuum with tar distillate.

Renewals.—Extensive repairs over practically the whole surface of the metaled or paved portion of the highway.

Repairs.—The restoration or mending of a considerable amount of the metaled or paved portion of the highway, but not usually of a majority of the surface area. More extensive than "Patching" but less so than "Renewals."

Resurfacing.—The renewal of the surface of the crust or pavement.

Road.—A highway outside of an urban district.

Road-Bed.—The natural foundation of a roadway.

Road Metal.—Broken stone, gravel, slag, or similar material used in road and pavement construction and maintenance.

Roadway.—That portion of a highway particularly devoted to the use of vehicles.

Rock Asphalt.—Sandstone or limestone naturally impregnated with asphalt.

Rock Asphalt Pavement.—A wearing course composed of broken or pulverized rock asphalt with or without the addition of other bituminous materials.

Sand.—Finely divided rock detritus the particles of which will pass a 10-mesh and be retained on a 200-mesh screen. Also see "Gravel."

Sand-Clay Road.—A roadway composed of an intimate mixture of sand and clay.

Scarify.—To loosen and disturb superficially.

Screen.—In laboratory work an apparatus, in which the apertures are circular, for separating sizes of material.

Screenings.—Broken rock of a size that will pass through a ½- to ¾-in. screen, depending on the character of the stone.

Seal Coat.—A final superficial application of bituminous material during construction to a bituminous pavement.

Setting Up.—As applied to bituminous material, the relative quick change which takes place after its application to a roadway, indicated by its hardening after cooling and exposure to atmospheric and traffic conditions, as opposed to the slower changes later occurring gradually and almost imperceptibly.

Shaping.—Trimming up and preparing a sub-grade preparatory to applying the first course of the road metal or artificial foundation.

Sheet-Asphalt Pavement.—One having a wearing course composed of asphalt cement and sand of predetermined grading, with or without the addition of fine material, incorporated together by mixing methods.

Sheet Pavement.—A pavement free from frequent joints such as would accompany small slabs or blocks, and which has an appreciable thickness (say, in excess of 1 in. on the average) for its wearing course.

Shoulders.—The portion of the highway between the edges of the road metal or pavement and the gutters, slopes, or watercourses.

Side Drain.—See "Drainage."

Sidewalk.—The portion of the highway reserved for pedestrians.

Sieve.—In laboratory work an apparatus, in which the apertures are square, for separating sizes of material.

Silt.—Naturally deposited fine earthy material, which will pass a 200-mesh sieve.

Slag.—‡ Fused or partly fused compounds of silica in combination with lime or other bases, resulting in secondary products from the reduction of metallic ores.

Spalls.—Fragments broken off by a blow, irregular in shape, and of sufficient size to be comparable to the original mass.

Squegee.—A tool with a rubber or leather edge for scraping or cleaning hard surfaces, or for spreading and distributing liquid material over and into the superficial interstices of roadways.

Squegee Coat.—An application by means of the squegee.

Stone Block Pavement.—One having a wearing course composed of stone blocks quite or nearly rectangular in shape.

Street.—A highway in an urban district.

Sub-Grade.—The upper surface of the native foundation on which is placed the road metal or the artificial foundation, in case the latter is provided.

Superficial Coat.—A light surface coat.

Surface Coat.—See "Carpet."

Surface Treatment.—Treating the finished surface of a roadway with bituminous material.

Surfacing.—(1) The crust or pavement. (2) Constructing a crust or pavement. (3) Finally finishing the surface of a roadway. (4) Treating the surface of a finished roadway with a bituminous material.

Tailings.—Stones which after going through the crusher do not pass through the largest openings of the screens.

Tar.—† Bitumen which yields pitch upon fractional distillation and which is produced as a distillate by the destructive distillation of bitumens, pyro-bitumens, or organic material.

Telford.—Properly an artificial foundation advocated by Thomas Telford (1757-1820), and consisting of a pavement of stone about 8 in. thick, laid by hand, and closely packed and wedged together. The individual stones were desired to be about 16 sq. in. in section, and about 8 in. in length. They were set close together on the prepared sub-grade, their longest dimension vertical and on their larger ends, their interstices chinked with smaller stones, and the whole rammed (or rolled) until firm and unyielding.

Telford Macadam.—Macadam with an artificial foundation of Telford.

Under-Drain.—See "Drainage."

Up-Keep.—Maintenance.

V-Drain.—See "Drainage."

Viscosity.—‡ The measure of the resistance to flow of a bituminous material, usually stated as the time of flow of a given quantity of the material through a given orifice.

Volatile.—Applied to those fractions of bituminous materials which will evaporate at climatic temperatures.

Water-Bound.—Bound or bonded with the aid of water.

Water-Gas Tars.—* Tars produced by cracking oil vapors at high temperatures in the manufacture of carburetted water-gas.

Wearing Coat.—The superficial layer of the crust or pavement exposed to traffic.

Wearing Course.—The course of the crust or pavement exposed to traffic.

Wood Block Pavement.—One having a wearing course composed of wood paving blocks, generally rectangular in shape.

The Committee wishes to express again its deep appreciation of the assistance rendered it by the Board of Direction and by your Secretary, Dr. Chas. Warren Hunt, as well as by members of the Society, and others.

Very respectfully,

For the Special Committee on Materials for Road Construction and on Standards for Their Test and Use.

ARTHUR H. BLANCHARD,
Secretary.

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OCTOBER 27TH, 1917.

APPENDIX A

FORM OF RECORD FOR DATA CONCERNING THE USE OF HIGHWAY MATERIALS

(The following forms of records are recommended for the use of Highway Engineers. They cover both bituminous and non-bituminous materials. Although combined in a single table, it is believed that the data for different kinds of road surfaces can advantageously be placed on separate sheets or cards for convenient filing and reference.)

General Information.

State............County.............Town or City................
Road or street...............Limits of improvement...............
Length of improvement, in feet..................................
Width of crust or pavement, in feet (average).......................
Area of crust or pavement, in square yards..........................
Kind of surface and foundation....................................
Percentage of grade, maximum—per cent....Minimum—per cent.....
Amount of crown, maximum..., minimum..., Nature of sub-grade....
Maximum and minimum air temperature during year................
Hours of working day.............Labor wage per hour............
Contractor ....
Dates of beginning and completion of improvement.................
Class of highway or nature of traffic..................................

Traffic Census for........ Hours, Being the Average of .... Observations Taken Between the Hours of........ and........ on...
Location of Point of Observation...........................

	Commercial Vehicles.			Passenger vehicles.
	Empty.	Loaded.	Estimate (in pounds) of maximum load per inch of tire.	
One-horse vehicles..............				
Two or three-horse vehicles......				
Four or more horse vehicles......				
Motor cycles....................				
" runabouts				
" touring cars (open or closed).				
" busses				
" trucks....................				

Construction and Cost Details.

A.—Foundation.

1. Material ....
2. Thickness ....
3. Cost per square yard.......................................
4. Estimated life, in years....................................

B.—Wearing Course.

1. Material ....
2. Thickness ....
3. Size of block or brick......................................
4. Kind and amount of bituminous cement.........................
5. Kind of joint..
6. Proportions of aggregate....................................
7. Cushion or binder course....................................
8. First cost per square yard...................................
9. Life, in years...
10. Average annual maintenance cost per square yard during life of wearing course ....

C.—Traffic Data.

1. Tons per year (2 000 lb.)...................................
2. Average tons per yard of width...............................
3. Proportion of tonnage on metal tires..........................
4. " " *C*-3 on tires 2 in. or less in width.................

Items of Cost for Each Square Yard.

	Foundation	Wearing Course
Materials..........................		
Labor..............................		
Superintendence....................		
Overhead, including interest on plant, depreciation, etc..................		

Data Per Square Yard.

A-3.—First cost of foundation......................................

A-5.—Annual interest and sinking fund for foundation............

A-6.—Total annual cost of foundation..............................

B-10.—Annual maintenance cost of wearing course.................

B-11.—Annual interest and sinking fund for wearing course..........

B-12.—Total annual cost of wearing course.........................

$$\text{Yearly cost per 1 000 tons of traffic} = \frac{A\text{-}6 + B\text{-}12}{C\text{-}2}\,1\,000 = \ldots\ldots\ldots$$

(The following data or such parts as apply to a particular road or street could be incorporated on the sheet or card containing the data

immediately preceding, or could be placed on a separate sheet or card containing the results of tests and analyses.)

Broken Stone and Broken Slag.

Name and origin........................ ....
Specific gravity........................ ....
Absorption of water per cubic foot........ ....
Abrasion, percentage of loss.............. ....
Toughness ....
Cementation ....
Crushing strength per square inch......... ....
Mechanical analysis..................... (Use table under this title)
Voids, percentage of, loose and compacted. ....

Gravel.

Location ....
Specific gravity........................ ....
Abrasion, percentage of loss.............. ....
Cementation ....
Mechanical analysis..................... (Use table under this title)
Voids, percentage of, loose and compacted. ....

Sand.

Location ....
Specific gravity........................ ....
Mechanical analysis..................... (Use table under this title)
Voids, percentage of, loose and compacted. ....
Tensile strength in cement briquettes, as compared with standard Ottawa sand... ....

Mixtures of Sand or Other Fine Highway Materials with Broken Stone, Broken Slag, or Gravel.

Specific gravity........................ ....
Mechanical analysis..................... (Use table under this title)
Voids, percentage of, loose and compacted. ....

Paving Brick.

Composition ....
Name of manufacturer.................... ....
Rattler test, percentage of loss.......... ....

Stone Block.

Name and origin........................ ....
Specific gravity........................ ....
Absorption of water per cubic foot...... ....
Abrasion, percentage of loss.............. ....
Toughness ....
Hardness ....
Crushing strength per square inch...... ....

Wood Block.

Character of wood....................	....
Weight of blocks......................	....
Soundness	....
Rings per radial inch..................	....
Quantity of preservative................	....
Absorption of water after treatment......	....
Character of preservative:	
Specific gravity at 25° cent. (77° Fahr.).	....
Specific gravity at 38° cent. (100° Fahr.)	....
Solubility in benzol or chloroform.....	....
Water content......................	....
Distillation:	
Up to 170° cent...................	....
170° to 200° cent...............	....
200° to 210° cent...............	....
210° to 235° cent...............	....
235° to 270° cent...............	....
270° to 300° cent...............	....
300° to 315° cent...............	....
315° to 355° cent...............	....

MECHANICAL ANALYSIS.

	Percentages by weight.		
Passing 200-mesh sieve........................	.....	.....	.....
“ 100 “ “ retained on 200-mesh sieve	.....	.....	.....
“ 80 “ “ “ “ 100 “ “	.....	.....	.....
“ 50 “ “ “ “ 80 “ “	.....	.....	.....
“ 40 “ “ “ “ 50 “ “	.....	.....	.....
“ 30 “ “ “ “ 40 “ “	.....	.....	.....
“ 20 “ “ “ “ 30 “ “	.....	.....	.....
“ 10 “ “ “ “ 20 “ “	.....	.....	.....
“ $\frac{1}{4}$-in. screen, “ “ 10 “ “	.....	.....	.....
“ $\frac{1}{2}$ “ “ “ “ $\frac{1}{4}$-in. screen	.....	.....	.....
“ $\frac{3}{4}$ “ “ “ “ $\frac{1}{2}$ “ “	.....	.....	.....
“ 1 “ “ “ “ $\frac{3}{4}$ “ “	.....	.....	.....
“ $1\frac{1}{4}$ “ “ “ “ 1 “ “	.....	.....	.....
“ $1\frac{1}{2}$ “ “ “ “ $1\frac{1}{4}$ “ “	.....	.....	.....
“ 2 “ “ “ “ $1\frac{1}{2}$ “ “	.....	.....	.....
“ $2\frac{1}{2}$ “ “ “ “ 2 “ “	.....	.....	.....
“ 3 “ “ “ “ $2\frac{1}{2}$ “ “	.....	.....	.....
“ $3\frac{1}{2}$ “ “ “ “ 3 “ “	.....	.....	.....
	.....	.....	.....

Portland Cement.

Loss on ignition, percentage............ ....
Insoluble residue, percentage........... ....
Specific gravity........................ ....
Retained on 200-mesh sieve, percentage.. ....
Retained on 100-mesh sieve, percentage.. ....
Steam test.............................. ....
Initial set, time, in minutes........... ....
Final set, time, in minutes............. ....
Tensile strength, neat, 24 hours........ ....
Tensile strength, neat, 7 days.......... ....
Tensile strength, 1:3 Ottawa sand, 7 days. ....
Tensile strength, 1:3 Ottawa sand, 28 days ....
Compressive strength, per square inch..... ....
Constancy of volume..................... ....

BITUMINOUS MATERIALS.

The following forms are given as illustrations of those to be used for recording the properties of bituminous materials.

Asphalt Cements for Bituminous Macadam, Bituminous Concrete, Asphalt Block and Sheet-Asphalt Pavements and Fillers for Brick and Stone Block Pavements.

Trade name...
Manufacturer ....
General characteristics................................
Specific gravity at 25° cent. (77° Fahr.)................ ....
Flash point.. ....
Solubility in CS_2 (carbon disulphide)................. ....
  Organic matter insoluble............................... ....
  Inorganic matter insoluble............................. ....
Solubility of bitumen in CCl_4 (carbon tetrachloride)...... ....
Solubility of bitumen in petroleum naphtha.............. ....
Penetration 4° cent. (39° Fahr.), 200 grammes, 1 min.... ....
Penetration 25° cent. (77° Fahr.), 100 grammes, 5 sec..... ....
Penetration 46° cent. (115° Fahr.), 50 grammes, 5 sec..... ....
Float test... ....
Melting point by ring and ball method.................. ....
Ductility at 4° cent. (39° Fahr.)...................... ....
Ductility at 25° cent. (77° Fahr.)..................... ....
Fixed carbon content................................... ....
Paraffin content.......................................
Loss on evaporation at 163° cent. (325° Fahr.), 5 hours.... ....
  Penetration of residue 4° cent. (39° Fahr.), 200 grammes, 1 min...
  Penetration of residue 25° cent. (77° Fahr.), 100 grammes, 5 sec....

Penetration of residue 46° cent. (115° Fahr.), 50 grammes, 5 sec.....
Melting point of residue, by ring and ball method.................
Float test on residue.......................................
Ductility of residue at 4° cent. (39° Fahr.)
Ductility of residue at 25° cent. (77° Fahr.)

Tar Cements for Bituminous Macadam
and Bituminous Concrete Pavements and Fillers
for Brick and Stone Block Pavements.

Trade name..
Manufacturer ....
General characteristics.....................................
Water ....
Specific gravity at 25° cent. (77° Fahr.)................ ....
Flash point.................................. ....
Solubility in CS_2 (carbon disulphide).................. ....
Specific viscosity, Engler.......................... ....
Melting point, by cube method...................... ....
Float test.................................... ....
Distillation by weight and by volume................... ....
Up to 110° cent.............................. ....
110° to 170° cent.............................. ....
170° to 235° cent.............................. ....
235° to 270° cent.............................. ....
270° to 300° cent.............................. ....
Specific gravity of total distillate at 25° cent. (77° Fahr.). ....
Melting point of residue, by cube method............... ....
Float test on residue.......................... ....

APPENDIX B

TESTS OF NON-BITUMINOUS MATERIALS.

It is recommended that the following methods for performing tests of non-bituminous materials be adopted as standards:

Specific Gravity of Coarse Aggregates.*

The apparent specific gravity shall be determined in the following manner:

The sample, weighing 1 000 grammes and composed of pieces approximately cubical or spherical in shape and retained on a screen having 1.27-cm. (½-in.) circular openings, shall be dried to constant weight at a temperature between 100 and 110° cent. (212 and 230° Fahr.), cooled, and weighed to the nearest 0.5 gramme. Record this weight as weight A. In the case of homogeneous material, the smallest particles in the sample may be retained on a screen having 1¼-in. circular openings.

Immerse the sample in water for 24 hours, surface-dry individual pieces with aid of a towel or blotting paper, and weigh. Record this weight as weight B.

Place the sample in a wire basket of approximately ¼-in. mesh, and about 12.7 cm. (5 in.) square and 10.3 cm. (4 in.) deep, suspend in water† from center of scale pan, and weigh. Record the difference between this weight and the weight of the empty basket suspended in water as weight C. (Weight of saturated sample immersed in water.)

The apparent specific gravity shall be calculated by dividing the weight of the dry sample (A) by the difference between the weights of the saturated sample in air (B) and in water (C), as follows:

$$\text{Apparent specific gravity} = \frac{A}{B - C}.$$

Attention is called to the distinction between apparent specific gravity and true specific gravity. Apparent specific gravity includes the voids in the specimen, and is, therefore, always less than or equal to, but never greater than, the true specific gravity of the material.

Apparent Specific Gravity of Sand, Stone Screenings, or Other Fine Highway Material.

Apparatus.—The determination shall be made with a Jackson specific gravity apparatus which shall consist of a burette, with graduations reading to 0.01 in specific gravity, about 23 cm. (9 in.) long and

* Proposed in 1917 by Committee D–4, "Standard Tests for Road Materials", of the Am. Soc. for Testing Materials.

† The basket may be conveniently suspended by a fine wire hung from a hook shaped in the form of a question mark with the top end resting on the center of the scale pan.

with an inside diameter of about 0.6 cm. (0.25 in.), which shall be connected with a glass bulb approximately 13 cm. (5.5 in.) long and 4.5 cm. (1.75 in.) in diameter, the glass bulb being of such size that from a mark on the neck at the top to a mark on the burette just below the bulb, the capacity is exactly 150 c.c. (5.07 oz.); and an Erlenmeyer flask which shall contain a hollow ground-glass stopper having the neck of the same bore as the burette and a capacity of exactly 200 c.c. (6.76 oz.) up to the graduation on the neck of the stopper.

Method of Determination.—The method shall consist of: First, dry at not more than 110° cent. (230° Fahr.) to a constant weight a sample weighing about 55 grammes; second, weigh to 0.1 gramme, 50 grammes of the dry sample and pour it into the unstoppered Erlenmeyer flask; third, fill the bulb and burette with kerosene, leaving just space enough to take the temperature by introducing a thermometer through the neck; fourth, remove the thermometer and add sufficient kerosene to fill exactly to the mark on the neck, drawing off any excess with the burette; fifth, run into the flask about one-half of the kerosene in the bulb to remove air bubbles and then run in more kerosene, removing any material adhering to the neck of the flask, until the kerosene is just below the ground glass; sixth, place the hollow ground-glass stopper in position and turn it to fit tightly, and then run in kerosene exactly to the 200-c.c. (6.76-oz.) graduation on the neck, care being taken to remove all air bubbles in the flask; seventh, read the specific gravity from the graduation on the burette, and the temperature of the oil in the flask, noting the difference between the temperature of the oil in the bulb before the determination and that of the oil in the flask after the determination; eighth, make a temperature correction to the reading of the specific gravity in accordance with the table furnished by the manufacturer of the apparatus, adding the correction if the temperature of the kerosene has increased and subtracting it if the temperature of the kerosene has decreased.

ABSORPTION OF WATER PER CUBIC FOOT OF ROCK.*

"The absorption of water per cubic foot of rock shall be determined by the following method: First, a sample weighing between 29 and 31 g. and approximately cubical in shape shall be dried in a closed oven for 1 hour at a temperature of 110° C. (230° F.) and then cooled in a desiccator for 1 hour; second, the sample shall be rapidly weighed in air; third, trial weighings in air and in water of another sample of approximately the same size shall be made in order to determine the approximate loss in weight on immersion; fourth, after the balances shall have been set at the calculated weight, the first sample shall be weighed as quickly as possible in distilled water having a temperature of 25° C. (77° F.); fifth, allow the sample to remain 48 hours in distilled

* Proposed in 1914 by Committee D-4, "Standard Tests for Road Materials", of the Am. Soc. for Testing Materials.

water maintained as nearly as practicable at 25° C. (77° F.), at the termination of which time bring the water to exactly this temperature and weigh the sample while immersed in it; sixth, the number of pounds of water absorbed per cubic foot of the sample shall be calculated by the following formula:

$$\text{Pounds of water absorbed per cubic foot} = \frac{W_2 - W_1}{W - W_1} \times 62.24,$$

in which W = the weight in grammes of sample in air, W_1 = the weight in grammes of sample in water just after immersion, W_2 = the weight in grammes of sample in water after 48 hours immersion, and 62.24 = the weight in pounds of a cubic foot of distilled water having a temperature of 25° C. (77° F.).

"Finally, the absorption of water per cubic foot of the rock, in pounds, shall be the average of three determinations made on three different samples according to the method above described."

Abrasion Test for Broken Stone or Broken Slag.*

"The machine shall consist of one or more hollow iron cylinders; closed at one end and furnished with a tightly fitting iron cover at the other; the cylinders to be 20 cm. [7.87 in.] in diameter and 34 cm. [13.38 in.] in depth inside. These cylinders are to be mounted on a shaft at an angle of 30° with the axis of rotation of the shaft.

"At least [13.6 kg.] 30 lb. of coarsely broken stone shall be available for a test. The rock to be tested shall be broken in pieces as nearly uniform in size as possible, and as nearly 50 pieces as possible shall constitute a test sample. The total weight of rock in a test shall be within 10 grammes of 5 kilogrammes [11.02 lb.]. All test pieces shall be washed and thoroughly dried before weighing. 10 000 revolutions, at the rate of between 30 and 33 to the minute, must constitute a test. Only the percentage of material worn off which will pass through a 0.16 cm. (1-16 inch) mesh sieve shall be considered in determining the amount of wear. * * *"

Abrasion Test for Gravel.

Note.—As tests to determine the loss on abrasion of gravel are in an experimental stage, the Committee has included two methods which have been used and are being investigated. In noting the results obtained, the Committee advises stating the method used.

Method No. 1.—The test for abrasion of gravel shall be made with a Deval abrasion machine. (See "Abrasion Test for Broken Stone or Broken Slag.")

A charge of gravel shall consist of pieces which shall pass a screen having circular openings 5.08 cm. (2 in.) in diameter and be retained on a screen having circular openings 1.27 cm. (½ in.) in diameter. The total weight of gravel in a charge shall be within 10 grammes of 5 kg. (11.02 lb.). The gravel to compose a charge shall be washed.

* Method adopted by the Am. Soc. for Testing Materials, [illegible]

and dried in a closed oven for 1 hour at a temperature within 5° of 110° cent. (230° Fahr.). The charge of gravel shall be placed in one cylinder of the machine, which shall be rotated at a rate of not less than 30 nor more than 33 rev. per min. Ten thousand revolutions shall constitute a test. The percentage of material worn off which will pass through a sieve having openings of 0.16 cm. ($\frac{1}{16}$ in.) shall be considered the amount of wear of the charge of gravel. The loss by abrasion, determined as stated, shall be expressed in terms of the percentage of the total weight of the charge of gravel.

*Method No. 2.**—The aggregate is first screened through screens having circular openings 2 in., 1 in., and ½ in. in diameter. The sizes used for this test are divided equally between those passing the 2-in. and retained on the 1-in. screen, and those passing the 1-in. and retained on the ½-in. screen. The material of these sizes is washed and dried. The following weights of the dried stone are then taken: 2 500 grammes of the size passing the 2-in. and retained on the 1-in. screen, and 2 500 grammes of the size passing the 1-in. and retained on the ½-in. screen. This material is placed in the cast-iron cylinder of the Deval machine, as specified for the standard abrasion on stone. Briefly described, this machine consists of a frame and two or more cylinders mounted at an angle of 30° with the axis of rotation. The cylinders are 20 cm. in diameter and 34 cm. deep, inside dimensions. Six cast-iron spheres, 1.875 in. in diameter and weighing approximately 0.95 lb. (0.45 kg.) each, are placed in the cylinder as an abrasive charge. (The iron composing these spheres is the same as that used for the spheres in the Standard Paving Brick Rattler Test.) After the cast-iron spheres have been placed in the cylinder the lid is bolted on and the cylinder is mounted in the frame of the Deval machine. The duration of the test and the rate of rotation are the same as specified for the standard test for stone, namely, 10 000 revolutions at a rate of from 30 to 33 rev. per min. At the completion of the test the material is taken out and screened through a 16-mesh sieve. The material retained on the sieve is washed and dried and the percentage of loss by abrasion of the material passing the 16-mesh sieve is calculated. When the material has a specific gravity of less than 2.20, a total weight of 4 000 grammes, instead of 5 000 grammes, shall be used in the abrasion test.

Toughness Test for Rock or Slag.†

Definition.—Toughness, as applied to rock, is the resistance offered to fracture under impact, expressed as the final height of blow required of a standard hammer to cause fracture of a cylindrical test specimen of given dimensions.

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* Proposed by the First Conference of State Highway Testing Engineers and Chemists held at the U. S. Office of Public Roads in February, 1917.

† Proposed in 1917 by Committee D-4, "Standard Tests for Road Materials", of the Am. Soc. for Testing Materials.

Sampling.—Quarry samples of rock from which test specimens are to be prepared shall measure at least 6 in. on a side and at least 4 in. in thickness, and, when possible, shall have the plane of structural weakness* of the rock plainly marked thereon. Samples should be taken from freshly quarried material, and only from pieces which show no evidences of incipient fracture due to blasting or other causes. The samples should preferably be split from large pieces by the use of plugs and feathers, and not by sledging. Commercial stone-block samples from which test specimens are to be prepared shall measure at least 3 in. on each edge.

Size and Form of Test Specimen.—Specimens for test shall be cylinders prepared as described in the next paragraph, 25 mm. in height and from 24 to 25 mm. in diameter. Three test specimens shall constitute a test set. The ends of the specimens shall be plane surfaces at right angles to the axis of the cylinder.

Preparation of Test Specimens.—One set of specimens shall be drilled perpendicular and another parallel to the plane of structural weakness of the rock, if such plane is apparent. If a plane of structural weakness is not apparent, one set of specimens shall be drilled at random. Specimens shall be drilled in a manner which will not subject the material to undue stresses and will insure the specified dimensions.† The ends of the cylinders may be sawed with a band or diamond saw,‡ or in any other way which will not induce incipient fracture, but shall not be chipped or broken off with a hammer. After sawing, the ends of the specimens shall be ground plane with carborundum or emery on a cast-iron lap until the cylinders are 24 mm. in length.

Impact Machine.—Any form of impact machine which will comply with the following essentials may be used in making the test: A cast-iron anvil weighing not less than 50 kg., firmly fixed on a solid foundation; a hammer weighing 2 kg., arranged so as to fall freely between suitable guides; a plunger made of hardened steel, and weighing 1 kg., arranged to slide freely in a vertical direction in a sleeve, the lower end of the plunger being spherical in shape, with a radius of 1 cm.; means for raising the hammer and for dropping it on the plunger from any specified height from 1 to not less than 75 cm., and means for determining the height of fall to approximately 1 mm.; means for holding the cylindrical test specimen securely on the anvil without rigid lateral support, and under the plunger in such a way that the center of its upper surface, throughout the test, shall be tangent to the spherical end of the plunger at its lowest point.

* The plane of structural weakness, in certain cases, may be the rift, cleavage, or bedding plane.

† The form of diamond drill described in *Bulletin No. 347*, U. S. Department of Agriculture, pp. 6–7, is recommended, and should prove satisfactory, if the instructions are strictly followed.

‡ A satisfactory form of diamond saw is described in *Bulletin No. 347*, U. S. Department of Agriculture, pp. 7–9.

Method of Testing.—The test shall consist of a 1-cm. fall of the hammer for the first blow, a 2-cm. fall for the second blow, and an increase of 1-cm. fall for each succeeding blow, until failure of the test specimen occurs.

Recording and Reporting Results.—The height of the blow, in centimeters, at failure shall be the toughness of the test specimen. The individual and the average toughness of three test specimens shall be reported when no plane of structural weakness is apparent. In cases where a plane of structural weakness is apparent, the individual and average toughness of the three specimens in each set shall be reported and identified. Any peculiar condition of a test specimen which might affect the result, such as the presence of seams, fissures, etc., shall be noted and recorded with the test result.

Hardness Test for Rock or Slag.

The test for hardness shall be made with a "Dorry", or similar machine, consisting of a revolving disk on which is fed, at a uniform rate, a standard quartz sand passing a 30- and retained on a 40-mesh sieve. Two cores, each 25 mm. (0.98 in.) in diameter, shall be cut from the material to be tested, and their faces ground off so as to be at right angles to the long axes of the cores. The cores shall be placed in the holders or dies and weighted so that the entire weight of each core with its holder and added weight is 1 250 grammes. Each core shall be ground in the machine on one face for 1 000 revolutions, after which it shall be reversed and ground on the other face for an equal number of revolutions. The loss of weight of each specimen shall be determined at the end of each 1 000 revolutions, and the average loss in weight shall be used for stating the hardness of the material, which latter shall be expressed by the formula: Hardness $= 20 - \frac{1}{3} W$, where W equals the average loss, in grammes per 1 000 revolutions.

Cementation of Rock, Slag, and Gravel Powders.

The cementation test shall be made as follows: Of the material to be tested, 500 grammes shall be broken to pass a 1.27-cm. ($\frac{1}{2}$-in.) mesh sieve and then placed in a ball mill with 90 c.c. (3.04 oz.) of water and two steel shot weighing together 9 kg. (20 lb.). The mill and its charge shall be revolved for $2\frac{1}{2}$ hours at a rate of 2 000 rev. per hour. The dough thus formed shall then be removed, and 25 grammes of an average sample of it shall be placed in a metal die, 25 mm. (0.98 in.) in diameter, and subjected to a pressure of 132 kg. per sq. cm. for an instant in a hydraulic press. The cylindrical briquette resulting should measure exactly 25 mm. (0.98 in.) in height. If it does not, subsequent samples of the dough shall be taken in such quantity that the resulting briquette after compression will be exactly

25 mm. (0.98 in.) in height. Five such briquettes shall be made and allowed to dry in the air for a period of 20 hours, after which they shall be heated for 4 hours in a hot-air oven at a temperature of 93.3° cent. (200° Fahr.), and then cooled in a desiccator for 20 min. These cylinders or briquettes shall then be tested in a machine, as follows:

The machine shall be arranged so that a 1-kg. (2.20-lb.) hammer is raised to a height of 1 cm. (0.39 in.) and then falls freely on a plunger transmitting the shock of the blows of the hammer through the plunger to the test piece, successive blows being struck by the hammer at a rate of 40 to 70 per min., until the test piece fails, which is indicated by the failure of the plunger or hammer to rebound. The test piece shall be placed on the anvil under the plunger without lateral support, and may be fastened in place on the anvil by a drop of shellac. The average of the number of blows on the five briquettes, required to produce failure in each case, is the result to be reported, and is the "coefficient of cementation."

Crushing Strength of Rock or Slag.

Cylinders shall be cut from a suitable block of the material to be tested, each of which cylinders shall be, as nearly as practicable, 5 cm. (2 in.) in diameter and 10 cm. (4 in.) in length. After cutting, the dimensions of each cylinder shall be accurately measured and recorded. Each cylinder shall then be subjected to compression, and the ultimate stress at which its failure occurs shall be noted. This stress divided by the average area in cross-section of the cylinder in square inches shall be reported. It is desirable that the test of the material shall be made on at least three such cylinders separately, and the average of the three or more specimens shall be taken as the average resistance to crushing of the material. In making the test, the cylinder shall be fixed in the testing machine so as to be unsupported on its sides and rest squarely on its ends, and the compressive stress shall be applied cumulatively. The ends of the cylinder shall be at right angles to its long axis, and the blocks or pieces of the machine in contact with the ends of the cylinder and through which the pressure is transmitted shall have such position and freedom of movement in the machine as will insure the application of the stress directly along or parallel to the long axis of the cylinder.

Mechanical Analysis of Broken Stone, Broken Slag, or Gravel.*

The method shall consist of, first, drying at not more than 110° cent. (230° Fahr.), to a constant weight, a sample weighing in pounds six times the diameter in inches of the largest holes required; second, passing the sample through such of the following sized screens having circular openings as are required or called for by the specification, the screens to be used in the order named: 8.89 cm. (3½ in.), 7.62 cm.

* Method adopted in 1916 by the Am. Soc. for Testing Materials.

(3 in.), 6.35 cm. (2½ in.), 5.08 cm. (2 in.), 3.81 cm. (1½ in.), 3.18 cm. (1¼ in.), 2.54 cm. (1 in.), 1.90 cm. (¾ in.), 1.27 cm. (½ in.), and 0.64 cm. (¼ in.); third, determining the percentage by weight retained on each screen; fourth, recording the mechanical analysis in the following manner:

Percentage passing 0.64-cm. (¼-in.) screen.................. =
Percentage passing 1.27-cm. (½-in.) screen and retained on 0.64-cm. (¼-in.) screen.................................. =
Percentage passing 1.90-cm. (¾-in.) screen and retained on 1.27-cm. (½-in.) screen.................................. =
Percentage passing 2.54-cm. (1-in.) screen and retained on 1.90-cm. (¾-in.) screen.................................. =
.... =

100.00

MECHANICAL ANALYSIS OF SAND OR OTHER FINE HIGHWAY MATERIAL.

The method shall consist of: First, drying at not more than 110° cent. (230° Fahr.) to a constant weight a sample weighing 50 grammes; second, passing the sample through each of the following mesh sieves, the sieves to be used in the order named:

Meshes per linear inch (2.54 cm.).	Diameter of Wire: Inches.	Diameter of Wire: Millimeters.
10	0.027	0.6858
20	0.0165	0.4191
30	0.01375	0.34925
40	0.01025	0.26035
50	0.009	0.22865
80	0.00575	0.1460
100	0.0045	0.1143
200	0.00235	0.05969

third, determining the percentage by weight retained on each sieve, the sifting being continued on each sieve until less than 1% of the weight retained on each sieve shall pass through the sieve during the last minute of sifting; fourth, recording the mechanical analysis in the following manner:

Percentage passing 200-mesh sieve........................ =
Percentage passing 100-mesh sieve and retained on 200-mesh sieve .... =
Percentage passing 80-mesh sieve and retained on 100-mesh sieve .... =
Percentage passing 50-mesh sieve and retained on 80-mesh sieve .... =
.... =

100.00

Mechanical Analysis of Mixtures of Sand or Other Fine Highway Material with Broken Stone, Broken Slag, or Gravel.

The method shall consist of: First, drying at not more than 110° cent. (230° Fahr.) to a constant weight, a sample weighing in pounds six times the diameter in inches of the largest holes required; second, separating the sample by the use of a 10-mesh sieve (American Society for Testing Materials standard sieve); third, examining the portion retained on the 10-mesh sieve in accordance with the method for making a "Mechanical Analysis of Broken Stone, Broken Slag, or Gravel"; fourth, examining the portion passing the 10-mesh sieve in accordance with the method for making a "Mechanical Analysis of Sand or Other Fine Highway Material"; fifth, recording the mechanical analysis in the following manner:

Percentage passing 200-mesh sieve........................ =
Percentage passing 100-mesh sieve and retained on 200-mesh sieve .... =
Percentage passing 80-mesh sieve and retained on 100-mesh sieve .... =
.... =
Percentage passing 10-mesh sieve and retained on 20-mesh sieve .... =
Percentage passing 0.64-cm. (¼-in.) screen and retained on 10-mesh sieve.................................... =
Percentage passing 1.27-cm. (½-in.) screen and retained on 0.64-cm. (¼-in.) screen............................ =
Percentage passing 1.90-cm. (¾-in.) screen and retained on 1.27-cm. (½-in.) screen............................ =
.... =

.... = 100.00

Voids in Mineral Aggregates.*

"The voids in mineral aggregates shall be determined by the Cone Specific Gravity Method. In the method of making the determination of voids, as hereinafter described, there shall be used a truncated cone made of No. 18, B. & S.-gauge galvanized steel with caulked seams, and having the following dimensions: over-all diameter of bottom, 25.4 cm. (10 in.); over-all height, 25.4 cm. (10 in.); inside diameter of opening, 7.6 cm. (3 in.). The test shall be made in the following manner: First, thoroughly mix the aggregate by rolling on paper; second, fill the cone with aggregates, avoiding segregation; third, compact aggregate in cone by oscillation on edge of cone resting on wooden floor, wooden box, or block of wood, and use cotton waste pressed against surface of aggregate to prevent segregation during oscillation;

* Proposed in 1915 by Committee D-4, "Standard Tests for Road Materials", of the Am. Soc. for Testing Materials.

fourth, continue to add aggregate and compact until the cone is full of thoroughly compacted aggregate, which process will require from 300 to 500 oscillations; fifth, weigh cone with aggregate; sixth, weigh cone empty; seventh, weigh cone full of clean water; eighth, determine the specific gravity of aggregate; ninth, the percentage of voids in the aggregate shall be calculated by the following formula:

$$\text{Percentage of voids} = \left(1 - \frac{C - A}{(B - A)\,D}\right) 100$$

in which A = the weight in grammes of the cone; B = the weight in grammes of the cone filled with water; C = the weight in grammes of the cone filled with compacted aggregate; D = the specific gravity of the aggregate."

Rattler Test for Paving Brick.*

Construction of the Rattler.

General Design.—The machine shall be of good mechanical construction, self-contained, shall conform to the following details of material and dimensions, and shall consist of barrel, frame, and driving mechanisms as herein described.

The Barrel.—The barrel of the machine shall be made up of the heads, head-liners, staves, and stave-liners.

The Frame and Driving Mechanism.—The barrel shall be mounted on a cast-iron frame of sufficient strength and rigidity to support it without undue vibration. It shall rest on a rigid foundation with or without the interposition of wooden plates, and shall be fastened thereto by bolts at not less than four points. It shall be driven by gearing having a ratio of driver to driven of not less than one to four.

The Abrasive Charge.—The abrasive charge shall consist of cast-iron spheres of two sizes. When new, the larger spheres shall be 9.52 cm. (3.75 in.) in diameter and shall weigh approximately 3.40 kg. (7.5 lb.) each. Ten spheres of this size shall be used. These shall be weighed separately after each ten tests, and if the weight of any large sphere falls to 3.175 kg. (7 lb.), it shall be discarded and a new one substituted; provided, however, that all the large spheres shall not be discarded and substituted by new ones at any single time, and that, so far as possible, the large spheres shall compose a graduated series in various stages of wear. When new, the smaller spheres shall be 4.762 cm. (1.875 in.) in diameter and shall weigh approximately 0.43 kg. (0.95 lb.) each. In general, the number of small spheres in a charge shall not fall below 245 nor exceed 260. The collective weight of the large and small spheres shall be as nearly 136 kg. (300 lb.) as possible. No small sphere shall be retained in use after it has been worn down so that it will pass a circular hole 4.45 cm. (1.75 in.) in diameter, drilled in an iron plate 0.64 cm. (¼ in.) in thickness, or weigh less than

* Adapted from the "Standard Specifications for Paving Brick", adopted in 1915 by the Am. Soc. for Testing Materials.

0.34 kg. (0.75 lb.). Further, the small spheres shall be tested, by passing them over the above plate or by weighing, after ten tests, and any which pass through or fall below the specified weight, shall be replaced by new spheres; provided, further, that all the small spheres shall not be rejected and replaced by new ones at any one time, and that, so far as possible, the small spheres shall compose a graduated series in various stages of wear. At any time that any sphere is found to be broken or defective, it shall at once be replaced.

The iron composing these spheres shall have a chemical composition within the following limits:

Combined carbon	Not less than		2.50	per	cent.
Graphitic carbon	Not more than		0.25	"	"
Silicon	"	" "	1.00	"	"
Manganese	"	" "	0.50	"	"
Phosphorus	"	" "	0.25	"	"
Sulphur	"	" "	0.08	"	"

Operation of the Test.

The Brick Charge.—The number of bricks per test shall be ten for all bricks of so-called "block-size", having dimensions which fall between 20.32 and 22.86 cm. (8 and 9 in.) in length, 7.62 and 9.52 cm. (3 and 3¾ in.) in breadth, and 9.52 and 10.8 cm. (3¾ and 4¼ in.) in thickness. No brick should be selected as part of a regular test that would be rejected by any other requirements of the specifications under which the purchase is made. (*Note by Committee.*—Each brick should be marked by small holes drilled in one of the faces of the brick, and the initial weight of each brick composing the charge should be determined.)

Speed and Duration of Revolution.—The rattler shall be rotated at a uniform rate of not less than 29.5 nor more than 30.5 rev. per min., and 1 800 revolutions shall constitute the test. A counting machine shall be attached to the rattler for recording the revolutions. A margin of not more than 10 revolutions will be allowed for stopping. Only one start and stop per test is generally acceptable. If, from accidental causes, the rattler is stopped and started more than once during a test, and the loss exceeds the maximum permissible under the specifications, the test shall be discarded and another made.

The Scales.—The scales shall have a capacity of not less than 136 kg. (300 lb.), shall be sensitive to 14.17 grammes (0.5 oz.), and shall be tested by a standard test weight at intervals of not less than every ten tests.

The Results.—The loss shall be calculated in percentage of the initial weight of the brick composing the charge. In weighing the rattled brick, any piece weighing less than 0.45 kg. (1 lb.) shall be

rejected. (*Note by Committee.*—The loss for each brick should also be calculated in percentage of the initial weight of each brick composing the charge.)

Absorption of Water by Wood Blocks After Treatment.

Five blocks of average character shall be heated in an oven to a temperature of 110° cent. (230° Fahr.) for 3 hours, then weighed, and immersed in water for the same length of time. At the end of this time, they shall be taken out, wiped dry, and weighed, the difference in weight before and after immersion, calculated on the weight after heating, being the percentage of absorption.

Cement.

For sampling, analysis, and testing of cement, the methods described in the "Final Report of the Special Committee on Uniform Tests of Cement" (*Transactions,* Am. Soc. C. E., Vol. LXXV, 1912, pp. 665 to 696) are recommended for use.

APPENDIX C

TESTS OF BITUMINOUS MATERIALS.

It is recommended that the following methods for performing tests of bituminous materials be adopted as standards:

Specific Gravity.

For liquid and semi-solid materials, some standard form of pyknometer shall be used. For solid materials, the suspension method shall be used. Material and distilled water shall have a temperature of 25° cent. (77° Fahr.).

The pyknometer to be used shall consist of a fairly heavy, straight-walled glass tube, 70 mm. (2.75 in.) long and 22 mm. (0.875 in.) in diameter, ground to receive a solid glass stopper with a hole of 1.6-mm. (0.063-in.) bore, in place of the usual capillary opening. The lower part of this stopper shall be made concave in order to allow all air bubbles to escape through the bore. The depth of the cup-shaped depression is 4.8 mm. (0.188 in.) at the center. The stoppered tube shall have a capacity of about 24 cu. cm. (0.811 oz.) and when empty shall weigh about 28 grammes. Its principal advantages are: (1) that any desired quantity of bituminous material may be poured in without touching the sides above the level desired; (2) it is easily cleaned; (3) on account of the 1.6-mm. (0.063-in.) bore, the stopper can be more easily inserted when the tube is filled with a very viscous oil than if it contained a capillary opening. When testing solid or semi-solid materials with the pyknometer, extreme care should be taken in melting, to avoid loss by evaporation, and, in filling the pyknometer, to avoid entrapping air.

When working with semi-solid bituminous materials which are too soft to be broken and handled in fragments, the following method of determining their specific gravity has been used with good results. The clean, dry pyknometer is first weighed empty and this weight is called a. It is then filled in the usual manner with freshly distilled water at 25° cent. (77° Fahr.), and the weight is again taken and called b. A small quantity of the material is then placed in a spoon and brought to a fluid condition by the gentle application of heat, with care that no loss by evaporation occurs. When sufficiently fluid, enough is poured into the dry pyknometer, which may also be warmed, to fill it about half full, without allowing the material to touch the sides of the tube above the desired level. The tube and contents are then allowed to cool to room temperature, after which the tube, with the stopper, is carefully weighed. This weight is called c. Distilled water, at 25° cent. (77° Fahr.), is then poured in until

the pyknometer is full. After this the stopper is inserted and the whole is cooled to 25° cent. (77° Fahr.) by a 30-min. immersion in a beaker of distilled water maintained at this temperature. All surplus moisture is then removed with a soft cloth, and the pyknometer and contents are weighed. This weight is called d. From the weights obtained, the specific gravity of the material may be readily calculated by the following formula:

$$\text{Specific gravity } \left.\begin{array}{r}25^\circ \text{ cent. } (77^\circ \text{ Fahr.})\\ 25^\circ \text{ cent. } (77^\circ \text{ Fahr.})\end{array}\right/ \Big\} = \frac{c - a}{(b - a) - (d - c)}.$$

Both a and b are constants, and need be determined only once. It is necessary, therefore, to make only two weighings for each determination after the first. Results obtained according to this method are accurate to within two units in the third decimal place, whereas the open-tube method commonly used is accurate to the second decimal place only.

The specific gravity of fluid bituminous material may be determined in the ordinary manner with this pyknometer by completely filling it with the material and dividing the weight of the bituminous material thus obtained by that of the same volume of water.

Flash Point.

The flash point shall be determined by the closed-cup test.

Although, for ordinary purposes, the open-cup method of determining the flash and burning points of bituminous materials is reasonably accurate, the closed-cup method described below is to be preferred.

The oil tester shall consist of a copper oil cup having a capacity of about 300 cu. cm. (10.1 oz.), and shall be heated in a water or oil bath by a small Bunsen flame. The cup shall be provided with a glass cover carrying a thermometer, and a hole for inserting the testing flame. The testing flame shall be obtained from a jet of gas passed through the piece of glass tubing, and shall be about 5 mm. (0.197 in.) in length.

The flash test shall be made as follows: The oil cup shall first be removed and the bath filled with water or cottonseed oil. The oil may always be used, and is necessary for bituminous material flashing at a temperature of more than 100° cent. (212° Fahr.). The oil cup shall be replaced and filled with the material to be tested to within 3 mm. (0.118 in.) of the flange joining the cup and the vapor chamber above. The glass cover shall then be placed on the oil cup and the thermometer adjusted so that its bulk shall be just covered by the bituminous material. The Bunsen flame shall be applied in such a manner that the temperature of the material in the

cup shall be raised at the rate of about 5° cent. (9° Fahr.) per min. From time to time the testing flame shall be inserted in the opening in the cover to about half way between the surface of the material and the cover. The appearance of a faint bluish flame over the entire surface of the bituminous material will show that the flash point has been reached and the temperature at this point is taken.

Solubility in Carbon Disulphide (CS_2).

This test shall consist in dissolving the bituminous material in carbon disulphide and recovering any insoluble matter by filtering the solution through an asbestos felt. The Gooch crucible used for the determination shall be 4.4 cm. (1.722 in.) wide at the top, tapering to 3.6 cm. (1.417 in.) at the bottom, and shall be 2.5 cm. (0.984 in.) deep.

The asbestos shall be cut with scissors into pieces not exceeding 1 cm. (0.394 in.) in length, after which it shall be shaken up with just sufficient water to pour easily. The crucible shall be filled with the suspended asbestos and allowed to settle for a few moments. A light suction shall then be applied to draw off all the water and leave a firm mat of asbestos in the crucible. More of the suspended material shall be added, and the operation shall be repeated until the felt shall be so dense that it scarcely transmits light when held so that the bottom of the crucible is between the eye and the source of light. The felt shall then be washed several times with water, and drawn firmly against the bottom of the crucible by an increased suction. The crucible shall be removed to a drying oven for a few minutes, after which it shall be ignited at red heat over a Bunsen burner, cooled in a desiccator, and weighed.

Two grammes of bituminous material or 10 grammes of an asphalt topping or rock asphalt shall then be placed in an Erlenmeyer flask, which shall have been weighed previously, and the accurate weight of the sample obtained. One hundred cubic centimeters (3.381 oz.) of chemically pure carbon disulphide shall be poured into the flask, in small portions, with continual agitation, until all lumps disappear and nothing adheres to the bottom. The flask shall then be corked and set aside for 15 min. to allow settlement of the insoluble material.

The weighed Gooch crucible containing the felt shall be set up over the dry pressure flask, and the solution of bituminous material in carbon disulphide shall be decanted through the felt without suction by gradually tilting the flask, with care not to stir up any precipitate that may have settled out. At the first sign of any sediment coming out, the decantation shall be stopped and the filter allowed to drain. A small quantity of carbon disulphide shall then be washed down the sides of the flask, after which the precipitate shall be brought upon the felt and the flask scrubbed, if necessary,

with a feather or "policeman", to remove all adhering material. The contents of the crucible shall be washed with carbon disulphide until the washings run colorless. Suction shall then be applied until there is practically no òdor of carbon disulphide in the crucible, after which the outside of the crucible shall be cleaned with a small quantity of the solvent. The crucible and contents shall be dried in the hot-air oven at 100° cent. (212° Fahr.) for about 20 min., cooled in a desiccator, and weighed. If any appreciable quantity of insoluble matter adheres to the flask, it shall also be dried and weighed, and any increase over the original weight of the flask shall be added to the weight of insoluble matter in the crucible. The total weight of insoluble material may include both organic and mineral matter. The former, if present, shall be burned off by ignition at a red heat until no incandescent particles remain, thus leaving the mineral matter or ash, which can be weighed on cooling. The difference between the total weight of material insoluble in carbon disulphide and the weight of substance taken equals the total bitumen, and the percentage weights are calculated and reported as total bitumen, and insoluble organic and inorganic matter, on the basis of the weight of material taken for analysis.

This method is quite satisfactory for straight oil and tar products, but, where native asphalts are present, it will be found practically impossible to retain all the finely divided mineral matter on an asbestos felt. It is generally more accurate, therefore, to obtain the result for total mineral matter by direct ignition of a 1-gramme sample in a platinum crucible, or to use the result for ash obtained in the fixed carbon test. The total bitumen is then determined by deducting from 100% the sum of the percentages of total mineral matter and insoluble organic matter. If the presence of a carbonate mineral is suspected, the percentage of mineral matter may be most accurately obtained by treating the ash from the fixed carbon determination with a few drops of ammonium carbonate solution, drying at 100° cent. (212° Fahr.), then heating for a few minutes at a dull red heat, cooling, and weighing again.

When difficulty in filtering is experienced—for instance, when Trinidad asphalt is present in any quantity—a longer period of subsidence than 15 min. is necessary, and the following method, adopted in 1911 by the American Society for Testing Materials is recommended:

Analysis of Sample.—After drying, from 2 to 15 grammes (as may be necessary to insure the presence of 1 to 2 grammes of pure bitumen) are weighed into a 150-cc. tared Erlenmeyer flask, and treated with 100 cc. of carbon disulphide. The flask is then loosely corked and shaken from time to time until all large particles of the material have been broken up. It is then set aside for 48 hours to settle. The

solution is decanted into a similar flask that has been previously weighed. As much of the solvent is poured off as possible without disturbing the residue. The contents of the first flask are again treated with fresh carbon disulphide, shaken as before, and then put away with the second flask for 48 hours to settle.

The liquid in the second flask is then carefully decanted on a weighed Gooch crucible, 3.2 cm. in diameter at the bottom, fitted with an asbestos filter, and the contents of the first flask are similarly treated. The asbestos filter is made of ignited long-fiber amphibole, packed in the bottom of a Gooch crucible to the depth of not more than ⅛ in. In filtering, no vacuum is to be used and the temperature is to be kept between 20° and 25° cent. After passing the liquid contents of both flasks through the filter, the residue on the filter is thoroughly washed, and the residues remaining in them are shaken with more fresh carbon disulphide and allowed to settle for 24 hours, or until it is seen that a good subsidation has taken place. The solvent in both flasks is then again decanted through the filter, and the residues remaining in them are washed until the washings are practically colorless. All washings are to be passed through the Gooch crucible.

The crucible and both flasks are then dried at 125° cent. and weighed. The filtrate containing the bitumen is evaporated, the bituminous residue burned, and the weight of the ash thus obtained added to that of the residue in the two flasks and the crucible. The sum of these weights deducted from the weight of substance taken gives the weight of soluble bitumen.

Solubility of Bitumen in Carbon Tetrachloride (CCl_4).

The test shall be conducted in exactly the same manner as described for the test for "Solubility in Carbon Disulphide", except that 100 cu. cm. (3.381 oz.) of chemically pure carbon tetrachloride shall be used in place of carbon disulphide, and the percentage of bitumen insoluble in carbon tetrachloride shall be reported on the basis of the bitumen taken as 100, the quantity of bitumen having been determined by the method described under the heading "Solubility in Carbon Disulphide."

Consistency.

The "Engler Viscosimeter," the "New York Testing Laboratory Float," or the "Penetrometer," shall be used, as practicable, at 4° cent. (39° Fahr.), 25° cent. (77° Fahr.), and 46° cent. (115° Fahr.).

Viscosity Test.

The viscosity of liquid bituminous materials shall be determined at any desired temperature by using the "Engler Viscosimeter." This apparatus consists of a brass vessel for holding the material to be

tested, and is closed by a cover. To the conical bottom is fitted a conical outflow tube exactly 20 mm. (0.787 in.) long, with a diameter of 2.9 mm. (0.114 in.) on top, and of 2.8 mm. (0.110 in.) on the bottom. This tube is closed and opened by a pointed hardwood stopper. Pointed metal projections are placed on the inside of the vessel at equal distances from the bottom, and serve for measuring the charge of material, which is 240 cu. cm. (8.116 oz.). A thermometer is used to ascertain the temperature of the material to be tested. The vessel is surrounded by a brass jacket, which holds the material which may be used as a heating bath, either water or cottonseed oil, according to the temperature at which the test is to be made. A tripod serves as a support for the apparatus, and also carries a ring burner by which the bath is heated directly. The measuring cylinder, having a capacity of 100 cu. cm. (3.381 oz.), which is sufficiently accurate for work with road materials, is placed directly under the outflow tube.

As all viscosity determinations should be compared with that of water at 25°, cent. (77° Fahr.), the apparatus shall have been previously calibrated as follows: The cup and outlet tube shall first be scrupulously cleaned. A piece of soft tissue paper is convenient for cleaning the tube. The stopper shall then be inserted in the tube, and the cup shall be filled with water at 25° cent. (77° Fahr.) to the top of the projections. The measuring cylinder shall be placed directly under the outflow tube so that the material, on flowing out, will not touch the sides. The stopper shall then be removed and the time required, both for 50 and 100 cu. cm. (1.691 and 3.381 oz.) to run out, shall be ascertained by using a stop-watch. The results thus obtained shall be checked a number of times. The time required for 50 cu. cm. (1.691 oz.) of water should be about 11 sec., and for 100 cu. cm. (3.381 oz.) about 22.8 sec.

Bituminous materials shall be tested in the same manner as water, and the temperature at which the test is made shall be controlled by the bath. The material shall be brought to the desired temperature and maintained there for at least 3 min. before making the test. The results are expressed as specific viscosity compared with water at 25° cent. (77° Fahr.), as follows:

$$\text{Specific viscosity at —— degrees centigrade for —— cu. cm.}$$
$$= \frac{\text{second for passage of given volume at —— degrees centigrade}}{\text{seconds for passage of same volume of water at 25° cent. (77° Fahr.)}}.$$

Float Test.

The float apparatus consists of two parts, an aluminum float or saucer and a conical brass collar. The two parts are made separately, so that one float may be used with a number of brass collars.

In making the test, the brass collar shall be placed with the small end down on the brass plate, which shall have been previously amalgamated with mercury by rubbing it first with a dilute solution of mercuric chloride or nitrate and then with mercury. A small quantity of the material to be tested shall be heated in the metal spoon until quite fluid, with care that it shall suffer no appreciable loss by volatilization and that it shall be kept free from air bubbles. It shall then be poured into the collar in a thin stream until slightly more than level with the top. After the material has cooled to room temperature, the surplus may be removed with a spatula blade which has been slightly heated. The collar and plate shall then be placed in one of the tin cups containing ice water maintained at 5° cent. (41° Fahr.), and left in this bath for 15 min. Meanwhile, the other cup shall be filled about three-fourths full of water and placed on the tripod, and the water shall be heated to any temperature desired for the test. This temperature shall be accurately maintained, and shall at no time throughout the entire test be allowed to vary more than 0.5° cent. (0.9° Fahr.) from the temperature selected. After the material to be tested has been kept in the ice water for 15 min., the collar and contents shall be removed from the plate and screwed into the aluminum float, which shall then be immediately floated in the warmed bath. As the plug of bituminous material becomes warm and fluid, it is gradually forced upward and out of the collar, until water gains entrance to the saucer and causes it to sink.

The time, in seconds, between placing the apparatus on the water and when the float sinks shall be taken as a measure of the consistency of the material under examination.

Penetration Test.*

Apparatus.—The container for holding the material to be tested shall be a flat-bottomed, cylindrical dish, 55 mm. ($2\frac{3}{16}$ in.) in diameter and 35 mm. ($1\frac{3}{8}$ in.) deep.* The needle for this test shall be a cylindrical steel rod 50.8 mm. (2 in.) long and having a diameter of 1.016 mm. (0.04 in.) and turned on one end to a sharp point having a 6.35-mm. ($\frac{1}{4}$-in.) taper. The water bath shall be maintained at a temperature not varying more than 0.1° cent. (0.18° Fahr.) from 25° cent. (77° Fahr.). The volume of water shall be not less than 10 liters, and the sample shall be immersed to a depth of not less than 10 cm. (4 in.) and shall be supported on a perforated shelf not less than 5 cm. (2 in.) from the bottom of the bath. Any apparatus which will allow the needle to penetrate without appreciable friction, and which is accurately calibrated to yield results in accordance with the definition of penetration, will be acceptable. The transfer dish for the container shall be a small dish or tray of such capacity as will

* Adopted in 1916 by the Am. Soc. for Testing Materials.

insure complete immersion of the container during the test. It shall be provided with some means which will insure a firm bearing and prevent rocking of the container.

Preparation of Sample.—The sample shall be completely melted at the lowest possible temperature, and stirred thoroughly until it is homogeneous and free from air bubbles. It shall then be poured into the sample container to a depth of not less than 15 mm. (⅝ in.). The sample shall be protected from dust and allowed to cool in an atmosphere not lower than 18° cent. (65° Fahr.) for 1 hour. It shall then be placed in the water bath along with the transfer dish and allowed to remain 1 hour.

Testing.—In making the test, the sample shall be placed in the transfer dish filled with water from the water bath of sufficient depth to cover the container completely. The transfer dish containing the sample shall then be placed on the stand of the penetration machine. The needle, loaded with specified weight (See Report Form for Asphalt Cement, Appendix A), shall be adjusted to make contact with the surface of the sample. This may be accomplished by making contact of the actual needle point with its image reflected by the surface of the sample from a properly placed source of light. Either the reading of the dial shall then be noted or the needle brought to zero. The needle is then released for the specified period of time, after which the penetration machine is adjusted to measure the distance penetrated. At least three tests shall be made at points on the surface of the sample not less than 1 cm. (⅜ in.) from the side of the container and not less than 1 cm. (⅜ in.) apart. After each test the sample and transfer dish shall be returned to the water bath, and the needle shall be carefully wiped toward its point with a clean, dry cloth, to remove all adhering bitumem. The reported penetration shall be the average of at least three tests the values of which shall not differ more than four points between maximum and minimum. When desirable to vary the temperature, time, and weight (See Report Form for Asphalt Cement, Appendix A), and, in order to provide for a uniform method of reporting results when variations are made, the samples shall be melted and cooled in air as above directed. They shall then be immersed in water or brine, as the case may require, for 1 hour at the temperature desired.

Melting Point.

Cube Method for Tar Cements.

The material under examination shall be first melted in a spoon by the gentle application of heat until sufficiently fluid to pour readily. Care shall be taken that it suffers no appreciable loss by volatilization. It shall then be poured into a 12.7-mm (0.5-in.) brass cubical mould, which shall have been amalgamated with mercury, and shall be placed

on an amalgamated brass plate. The brass may be amalgamated by washing it first with a dilute solution of mercuric chloride or nitrate, after which the mercury is rubbed into the surface. By this means the bituminous material is, to a considerable extent, prevented from sticking to the sides of the mould. The hot material shall slightly more than fill the mould, and, when cooled, the excess shall be cut off with a hot spatula.

After cooling to room temperature, the cube shall be removed from the mould and fastened on the lower arm of a No. 10 wire (B. & S. gauge), bent at right angles at one end and suspended beside a thermometer in a covered Jena glass beaker having a capacity of 400 cu. cm. (13.526 oz.), which shall be placed in a water bath, or, for high temperatures, a cottonseed-oil bath. The wire shall be passed through the center of two opposite faces of the cube, which shall then be suspended with its base 25.4 mm. (1 in.) above the bottom of the beaker. The water or oil bath shall consist of an 800-cu. cm. (27.051-oz.) low-form Jena glass beaker, suitably mounted for the application of heat from below. The beaker in which the cube is suspended shall be of the tall-form Jena type, without lip. The metal cover shall have two openings. A cork, through which passes the long arm of the wire, shall be inserted in one hole and the thermometer in the other. The bulb of the thermometer shall be just level with the cube and at an equal distance from the side of the beaker. In order that a reading of the thermometer may be made, if necessary, at the point which passes through the cover, the hole shall be triangular and covered with an ordinary object glass through which the stem of the thermometer may be seen. Readings made through this glass shall be calibrated to the angle of observation, which may be made constant by sighting always from the front edge of the opening to any given point on the stem of the thermometer below the cover.

After the test specimen shall have been placed in the apparatus, the liquid in the outer vessel shall be heated in such a manner that the thermometer registers an increase of 5° cent. (9° Fahr.) per min. The temperature at which the bituminous material touches a piece of paper placed in the bottom of the beaker shall be taken as the melting point. Determinations made in the manner described shall not vary more than 2° cent. (3.6° Fahr.) for successive trials on the same material. At the beginning of this test the temperature of both bituminous material and bath shall be approximately at 25° cent. (77° Fahr.).

Ring and Ball Method for Asphalt Cements.*

The apparatus shall consist of a brass ring, 15.875 mm. ($\frac{5}{8}$ in.) in diameter, 6.35 mm. ($\frac{1}{4}$ in.) deep, 2.38125 mm. ($\frac{3}{32}$ in.) wide, sus-

*Proposed in 1916 by Committee D-4, "Standard Tests for Road Materials", of the Am. Soc. for Testing Materials.

pended 25.40 mm. (1 in.) above bottom of a beaker; a steel ball, 9.525 mm. ($\frac{3}{8}$ in.) in diameter, weighing between 3.45 and 3.50 grammes; a standardized thermometer; a glass beaker, approximately 600-cc. capacity.

Carefully melt the sample and fill the ring with material to be tested. Remove any excess. Place ball in center of ring and suspend in beaker containing approximately 400 cc. of water at a temperature of 5° cent. (41° Fahr.). Arrange thermometer bulb within $\frac{1}{2}$ in. of sample and at same level. Apply heat uniformly over bottom of beaker in quantity sufficient to raise temperature 5° cent. (9° Fahr.) per min. Record temperature at starting test and every minute thereafter until test is completed. The rate of heating is very important. Softening point is temperature at which specimen has dropped 1 in. Successive tests should average within 3° cent. For temperatures above 95° cent., glycerine shall be used instead of water.

Loss on Evaporation.*

The amount lost by oils and asphaltic compounds when they are heated in an oven at a temperature of 163° cent. (325° Fahr.) plus or minus 1° cent. (2° Fahr.) shall be determined by heating 50 grammes of the water-free substance contained in a flat-bottomed dish, the inside dimensions of which are approximately $2\frac{3}{16}$ in. in diameter and $1\frac{3}{8}$ in. deep (3-oz. Gill style ointment box, deep style) for 5 hours. The oven in which the substance is heated shall be brought to the prescribed temperature before the sample is introduced, and the temperature of the sample under test shall be regarded as that of a similar quantity of the same material immediately adjoining it in the oven, in which the bulb of a standardized thermometer is immersed. The oven may be either of circular or rectangular form, and the source of heat either gas or electricity. The samples under test shall rest in the same relative position in a single row on a perforated circular shelf, $9\frac{3}{4}$ in. in diameter, suspended by a vertical shaft midway in the oven, which is revolved by mechanical means at the rate of from 5 to 6 rev. per min. (Note.—If additional periods of heating are desired, it is recommended that they be made in successive increments of 5 hours each.) If the residue after heating is to be tested for penetration, the sample should be thoroughly mixed by stirring until it is cool, and thereafter manipulated in accordance with the directions of the standard test for penetration of bituminous materials.

Distillation.*

Note.—Equivalents in English units have been added by the Special Committee on Materials for Road Construction.

* Adopted in 1916 by the Am. Soc. for Testing Materials.

Sampling.—The sample as received shall be thoroughly stirred and agitated, warming, if necessary, to insure a complete mixture before the portion for analysis is removed.

Dehydration.—If the presence of water is suspected, or known, the material shall be dehydrated before distillation. About 500 cu. cm. (16.907 oz.) of the material is placed in an 800-cu. cm. (27.051-oz.) copper still provided with a distilling head connected with a water-cooled condenser. A ring burner is used, starting with a small flame at the top of the still, and gradually lowering it, if necessary, until all the water has been driven off. The distillate is collected in a 200-cu. cm. separatory funnel with the tube cut off close to the stop-cock. When all the water has been driven over and the distillate has settled out, the water is drawn off and the oils are returned to the residue in the still. The contents of the still shall have cooled to below 100° cent. (212° Fahr.) before the oils are returned, and they shall be well stirred and mixed with the residue.

Apparatus.—The apparatus shall consist of the following standard parts:

(*a*) *Flask.*—The distillation flask shall be a 250-cu. cm. Engler distilling flask, having the following dimensions:

Diameter of bulb	8.0 cm.	(3.150 in.)
Length of neck	15.0 "	(5.906 ")
Diameter of neck	1.7 "	(0.670 ")
Surface of material to lower side of tubulature	11.0 "	(4.331 ")
Length of tubulature	15.0 "	(5.906 ")
Diameter of tubulature	0.9 "	(0.354 ")
Angle of tubulature	75°	

A variation of 3% from the foregoing measurements will be allowed.

(*b*) *Thermometer.*—The thermometer shall conform to the following requirements:

It shall be made of thermometric glass of a quality equivalent to suitable grades of Jena or Corning make. It shall be thoroughly annealed. It shall be filled above the mercury with inert gas which will not act chemically on or contaminate the mercury. The pressure of the gas shall be sufficient to prevent separation of the mercury column at all temperatures of the scale. There shall be a reservoir above the final graduation large enough so that the pressure will not become excessive at the highest temperature. The thermometer shall be finished at the top with a small glass ring or button suitable for attaching a tag. Each thermometer shall have for identification the maker's name, a serial number, and the letters "A. S. T. M. Distillation."

The thermometer shall be graduated from 0 to 400° cent. at intervals of 1° cent. Every fifth graduation shall be longer than the inter-

mediate ones, and every tenth graduation beginning at zero shall be numbered. The graduation marks and numbers shall be clear-cut and distinct.

The thermometer shall conform to the following dimensions:

Total length, maximum......	385 mm.	
Diameter of stem...........	7 "	; permissible variation, 0.5 mm.
Diameter of bulb, minimum..	5 "	; and shall not exceed diameter of stem.
Length of bulb.............	12.5 "	permissible variation, 2.5 mm.
Distance from 0° to bottom of bulb	30 "	; " " 5 "
Distance from 0° to 400°...	295 "	; " " 10 "

The accuracy of the thermometer when delivered to the purchaser shall be such that when tested at full immersion the maximum error from 0 to 200° cent. shall not exceed the following:

From	0 to 200° cent..................	0.5° cent.
"	200 " 300° cent..................	1.0° cent.
"	300 " 375° cent..................	1.5° cent.

The sensitiveness of the thermometer shall be such that when cooled to a temperature of 74° cent. below the boiling point of water at the barometric pressure, at the time of test, and plunged into free flow of steam, the meniscus shall pass the point 10° cent. below the boiling point of water in not more than 6 sec.

The thermometer shall be set up as for the distillation test, using water, naphthalene, and benzophenone as distilling liquids. The correctness of the thermometer shall be checked at 0 and 100° cent. after each third distillation until seasoned.

(*c*) *Condenser.*—The condenser tube shall have the following dimensions:

Length	500 mm.	(19.685 in.)
Width ....12 to	15 "	(0.472 to 0.591 in.)
Width of adaptor end....20 "	25 "	(0.787 " 0.984 ")

(*d*) *Stands.*—Two iron stands shall be provided, one with a universal clamp for holding the condenser, and one with a light grip arm with a cork-lined clamp for holding the flask.

(*e*) *Burner and Shield.*—A Bunsen burner shall be provided, with a tin shield, 20 cm. (7.784 in.) long and 9 cm. (3.543 in.) in diameter. The shield shall have a small hole through which to observe the flame.

(*f*) *Cylinders.*—The cylinders used in collecting the distillate shall have a capacity of 25 cu. cm. (0.845 oz.), and shall be graduated in tenths of a cubic centimeter.

Setting up the Apparatus.—The apparatus shall be set up, the thermometers being placed so that the top of the bulb is opposite the middle of the tubulature. All connections shall be tight.

Method.—One hundred cubic centimeters (3.381 oz.) of the dehydrated material to be tested shall be placed in a tared flask and weighed. After adjusting the thermometer, shield, condenser, etc., the distillation is commenced, the rate being regulated so that 1 cu. cm. (0.034 oz.) passes over every minute. The receiver is changed as the mercury column just passes the fractionating point.

	Up to	110° cent.	(230° Fahr.)
110° cent.	"	170° "	(338° ")
170° cent.	"	235° "	(455° ")
235° cent.	"	270° "	(518° ")
270° cent.	"	300° "	(572° ")

To determine the quantity of residue, the flask is weighed again when distillation is complete. During the distillation the condenser tube shall be warmed when necessary, in order to prevent the deposition of any sublimate. The percentages of fraction should be reported, both by weight and by volume.

Ductility.

A briquette of the material to be tested shall be formed by pouring the molten material into a briquette mould. The dimensions of the briquette shall be: 1 cm. (0.394 in.) in thickness throughout its entire length; distance between the clips or end pieces, 3 cm. (1.181 in.); width of asphalt cement section at mouth of clips, 2 cm. (0.787 in.); width at minimum cross-section, half way between clips, 1 cm. (0.394 in.). The center pieces are removable, the briquette mould being held together during moulding with a clamp or wire.

The moulding of the briquette shall be done as follows: The two center sections shall be well amalgamated to prevent the asphalt cement from adhering to them, and the briquette mould shall then be placed on a freshly amalgamated brass plate. The asphalt cement to be tested, while in a molten state, shall be poured into the mould, a slight excess being added to allow for shrinkage on cooling. When the asphalt cement in the mould is nearly cool, the briquette shall be cut off level, with a warm knife or spatula. When it is thoroughly cooled to the temperature at which it is desired to make the test, the clamp and the two side pieces are removed, leaving the briquette of asphalt cement held at each end by the ends of the mould, which now play the part of clips. The briquette shall be kept in water for 30 min. at 4° cent. (39° Fahr.) or 25° cent. (77° Fahr.) before testing, dependent on the temperature at which the ductility is desired. The briquette with the clips attached shall then be placed in a "ductility test machine" filled

with water at one of the above temperatures to a sufficient height to cover the briquette not less than 50 mm. (1.969 in.). This machine consists of a rectangular water-tight box, having a movable block working on a worm-gear from left to right. The left clip is held rigid by placing its ring over a short metal peg provided for this purpose; the right clip is placed over a similar rigid peg on the movable block. The movable block is provided with a pointer which moves along a centimeter scale. Before starting the test, the centimeter scale is adjusted to the pointer at zero. Power is then applied by the worm-gear pulling from left to right at the uniform rate of 5 cm. (1.969 in.) per min. The distance, in centimeters, registered by the pointer on the scale at the time of rupture of the thread of asphalt cement shall be taken as the ductility of the asphalt cement.

Solubility in Petroleum Naphtha.

Two grammes of the material shall be placed in a 4-oz. oil-sample bottle, made up to 100 cu. cm. (3.381 oz.) with 88° Baumé petroleum naphtha (boiling point between 40° cent. (104° Fahr.) and 55° cent. (131° Fahr.)), and the whole well shaken until the sample is digested. The bottle shall then be centrifugalized for 10 min., 50 cu. cm. (1.691 oz.) withdrawn into a weighed flask, the naphtha distilled off by a water bath, and the residue weighed. From this weight the percentage of solubility shall be calculated.

Fixed Carbon.

One gramme of the bituminous material shall be placed in a platinum crucible, weighing between 20 and 30 grammes, between 28 and 38 mm. (1.102 and 1.496 in.) in height, and having a tightly fitting cover provided with a flange, about 4 mm. (0.157 in.) in depth. The crucible and its contents shall then be heated, first gently and then more severely, until no smoke or flame shall issue between the crucible and the lid. It shall then be placed in the full flame of a Bunsen burner for 7 min., holding the cover down with the end of a pair of tongs until the most volatile products shall have been burned off. The crucible shall be supported on a platinum triangle with the bottom from 6 to 8 cm. (2.362 to 3.150 in.) above the top of the burner. The flame shall be fully 20 cm. (7.874 in.) high when burning free, and the determination shall be made in a place free from drafts. The upper surface of the cover shall burn clear, but the under surface may or may not be covered with carbon, dependent on the character of the bituminous material. The crucible shall be removed to the desiccator, and, when cool, shall be weighed, after which the cover shall be removed and the crucible placed in an inclined position over the Bunsen burner and ignited until nothing but ash remains. Any carbon deposited on the cover shall also be burned off. The weight of ash

remaining shall be deducted from the weight of the residue after the first ignition of the sample. The resulting weight is that of the fixed carbon, which shall be calculated on the basis of the total weight of the sample, exclusive of mineral matter.

Paraffin.

One hundred grammes of the material shall be distilled rapidly in a retort to a dry coke. Five grammes of the distillate shall then be thoroughly mixed in a 60-cu. cm. (2.029-oz.) flask with 25 cu. cm. (0.845 oz.) of Squibbs' absolute ether. Twenty-five cu. cm. (0.845 oz.) of Squibbs' absolute alcohol shall then be added, and the flask packed closely in a freezing mixture of finely crushed ice and salt for at least 30 min. The precipitate shall be filtered out quickly with a suction pump, using a No. 575 C. S. and S. 9-cm. hardened filter paper. The flask and precipitate shall then be rinsed and washed with a mixture of equal parts of Squibbs' alcohol and ether cooled to — 17° cent. (1° Fahr.) until free from oil (50 cu. cm. (1.691 oz.) of washing solution is usually sufficient). When sucked dry, the filter paper shall be removed and the waxy precipitate transferred to a small glass disk and evaporated on a steam bath. The residue (paraffin) remaining on the disk shall be weighed, and from this weight the percentage on the original 5-gramme sample shall be calculated.

Specific Gravity at 38° Cent. of Wood Block Preservative.*

A standardized hydrometer shall be used. A set of two with ranges 1.00 to 1.08, and 1.07 to 1.15 will suffice. Before taking the specific gravity, the oil in the cylinder should be stirred thoroughly with a glass rod, and this rod when withdrawn from the liquid should show no solid particles at the instant of withdrawal. Care should be taken that the hydrometer does not touch the sides or bottom of the cylinder when the reading is taken, and that the oil surface is free from froth and bubbles. If the specific gravity is determined at a higher temperature than desired, correction should be made by adding 0.0008 to the reading for each degree centigrade excess of temperature.

Solubility in Benzol or Chloroform of Wood Block Preservative.

From 5 to 10 grammes of the water-free oil is weighed out into a weighed 100-c.c. (3.38-oz.) beaker; 50 c.c. (1.69 oz.) of the solvent is added, and the solution is passed through a weighted 9-cm., C. S. and S., No. 575 filter paper in a short-stemmed funnel, the filtrate being passed into the flask to be subsequently used for the hot extraction. The beaker is washed clean from all soluble matter, dried, and weighed. The funnel, with filter paper and contents, is then placed in a N. Y. T. L. or Underwriter's form of glass extraction apparatus, and heat

* Modification of method proposed in 1915 by Committee D-7 on "Standard Specifications for Timber", of the Am. Soc. for Testing Materials.

is applied from a water-bath or hot plate until the extraction is complete and the filtrate runs through colorless. The filter and contents are then dried and weighed. The increase in weight is added to the increase in weight of the beaker, if any, the result being the weight of the insoluble matter. The weight of the insoluble matter thus found is subtracted from the weight of the material taken for analysis. The difference in weight is the weight of the soluble matter, from which the percentage is calculated.

Water Content of Wood Block Preservative.

From 250 to 300 c.c. (8.45 to 10.14 oz.) of the oil is weighed out into a 500-c.c. glass retort, or into a small copper still, provided with a distilling head. Heat is applied with a ring burner, starting with a small flame at the top of the still, and gradually lowering it until all the water has been driven off. The distillate of oil and water is collected in a graduated separatory funnel, the volume of water, in cubic centimeters, is read, and its percentage computed by volume. The water is then drawn off and the oils are returned to the residue in the still. The contents of the still shall have cooled to below 100° cent. (212° Fahr.) before the oils are returned, and they shall be well stirred and mixed with the residue.

Distillation Test for Wood Block Preservative.*

Apparatus for Distillation Test.

Retort.—This shall be a tabulated Jena glass retort of the usual form, with a capacity of 250 to 290 c.c. (8.45 to 9.8 oz.). The capacity shall be measured by placing the retort with the bottom of the bulb and the end of the offtake in the same horizontal plane, and pouring water into the bulb through the tubulature until it overflows the offtake. The quantity remaining in the bulb shall be considered as its capacity.

Shield.—An asbestos shield shall be used to protect the retort from air currents and to prevent radiation. This may be covered with galvanized iron, as such an arrangement is more convenient and more permanent.

Receivers.—Erlenmeyer flasks of from 50 to 100 c.c. capacity are of the most convenient form.

Thermometer.—The thermometer shall be of glass, well annealed, and shall undergo no serious change at the zero point when heated up to 400° cent. The space above the mercury column shall be filled with gas, either carbon dioxide or nitrogen, and the thermometer shall have an expansion chamber at the top. The scale shall read from 0 to 400° cent., in graduations of 1° cent., which shall be etched on the stem. The tip of the thermometer shall carry a ring for the

* Modification of method proposed in 1915 by Committee D-7 on "Standard Specifications for Timber", of the Am. Soc. for Testing Materials.

purpose of attaching tags. The thermometer shall have the following dimensions:

Total length, 375 mm.; tolerance, 10 mm.
Bulb length, 14 mm.; tolerance, 1 mm.
Distance from zero mark to bottom of bulb, 30 mm.; tolerance, 4 mm.
Scale length from zero mark to 400° cent., 295 mm.; tolerance, 5 mm.
Diameter of stem, 7 mm.; tolerance, 1 mm.
Diameter of bulb, 6 mm.; tolerance, 1 mm.

When standardized, the accuracy of such standardization should be as follows:

Up to 200°	cent.	.........to the nearest	0.5° cent.
200 to 300°	"	.........." " "	1.0° "
300 to 360°	"	.........." " "	1.5° "

Assembling for Distillation Test.

The retort shall be supported on a tripod or rings over two sheets of 20-mesh gauge, 15.24 cm. (6 in.) square. It shall be connected to the condenser tube by a tight cork joint. The thermometer shall be inserted through a cork in the tubulature, with the bottom of the bulb 1.27 cm. ($\frac{1}{2}$ in.) from the surface of the oil in the retort. The exact location of the thermometer bulb shall be determined by placing a vertical rule, graduated in divisions not exceeding 0.16 cm. ($\frac{1}{16}$ in.), back of the retort when the latter is in position for the test, and sighting the level of the liquid and the point for the bottom of the thermometer bulb. The distance from the bulb of the thermometer to the outlet end of the condenser tube shall be not more than 60.96 cm. (24 in.) nor less than 50.8 cm. (20 in.). The burner should be protected from drafts by a suitable shield or chimney.

Distillation Test.

Exactly 100 grammes of oil shall be weighed into the retort, the apparatus shall be assembled, and heat applied. The distillation shall be conducted at the rate of at least one drop, and not more than two drops, per second, and the distillate collected in weighed receivers. The condenser tube shall be warmed whenever necessary, to prevent accumulation of solid distillates. Fractions shall be collected at the following points:

Up to 170° cent. (338° Fahr.), 170-200° cent. (338-392° Fahr.), 200-210° cent. (392-410° Fahr.), 210-235° cent. (410-455° Fahr.), 235-270° cent. (455-518° Fahr.), 270-300° cent. (518-572° Fahr.), 300-315° cent. (572-599° Fahr.), 315-355° cent. (599-671° Fahr.).

The receivers shall be changed as the mercury passes the dividing temperature for each fraction. The last receiver shall be removed at

355° cent. (671° Fahr.), and drainage from the condenser, etc., shall not be considered as part of the fraction. For weighing the receivers and fractions, a balance accurate to at least 0.05 gramme shall be used. During the progress of the distillation the thermometer shall remain in its original position. No correction shall be made for the emergent stem of the thermometer.

When any measurable quantity of water is present in the distillate, it shall be separated as nearly as possible and reported separately, all results being calculated on a basis of dry oil. When more than 2% of water is present, water-free oil shall be obtained by separately distilling a larger quantity, returning any oil carried over with the water, and using dried oil for the final distillation. A copper tar still is a convenient implement for obtaining water-free oil.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

FINAL REPORT OF THE SPECIAL COMMITTEE ON STEEL COLUMNS AND STRUTS*

The Special Committee, authorized by vote of this Society, "to consider and report upon the design, ultimate strength, and safe working values of steel columns and struts", was organized in January, 1909, with eleven members, as follows:

ALFRED P. BOLLER,	FRANK C. OSBORN,
AUSTIN L. BOWMAN,	GEORGE H. PEGRAM,
EMIL GERBER,	LEWIS D. RIGHTS,
CHARLES F. LOWETH,	GEORGE F. SWAIN,
RALPH MODJESKI,	EMIL SWENSSON,

J. R. WORCESTER.

Austin L. Bowman was elected Chairman, and Lewis D. Rights, Secretary.

Since that time, Alfred P. Boller, Emil Gerber, and Austin L. Bowman have died. They had brought ripe experience and enthusiastic interest to the work of the Committee, and its members desire at this time to record their appreciation of the invaluable work which these three gentlemen did while they were with us. James H. Edwards, Rudolph P. Miller, and C. W. Hudson were appointed to fill vacancies. Frank C. Osborn and Rudolph P. Miller have resigned from the Committee, on account of the pressure of other important duties. George H. Pegram, who served as Chairman after the death of Austin L. Bowman, resigned from the Committee upon being elected President of the Society, but has continued to take an interest in the work as an *ex officio* member. Your Committee now consists of eight members, as will be seen by the list at the end of this report.

* To be presented to the Annual Meeting, January 16th, 1918.

During 1909 the Committee collected literature on the subject of column tests, and tabulated and diagrammed the records of tests on columns with cross-sections of more than 5 sq. in. in area. This information was presented as a Progress Report to the Annual Meeting in 1910.* The report expressed the opinion that these data were not adequate, and that further tests were necessary.

During 1909 Congress authorized the construction of a large testing machine at Washington, and the Bureau of Standards, Dr. S. W. Stratton, Director, offered to make a series of tests on columns as soon as the machine was completed, the expense to be borne by the United States Government. The Committee and Dr. Stratton conferred as to the nature of the tests, and the Committee outlined the programme of a series of nine types of test columns, Figs. 1, 2, and 3. Owing to delays in constructing the machine, and difficulties in securing the special steel, the testing of the columns did not begin until January, 1914.

So as to provide a permanent record of the methods used, the Committee republishes below portions of the Progress Report of 1915, as follows:

Testing Machine and Instruments.

All the tests have been made at the Bureau of Standards, with an Emery testing machine, which was authorized by Congress on March 4th, 1909, and was accepted in 1913. Its capacity is as follows:

Tension	1 150 000 lb.
Compression	2 300 000 lb.
Extreme length between heads:	
Tension	34 ft. 4 in.
Compression	33 ft. $1\frac{3}{4}$ in.
Clearance between screws	4 ft. 6 in.
Length of stroke of ram	5 ft.

It will be seen from Fig. 4 that it has two heads, one fixed and the other movable, on two straining screws, each 12 in. in diameter. The hydraulic ram which furnishes the pressure is in the movable head, and, as the stroke is only 5 ft., it is necessary to adjust this head for columns of different lengths. The weighing is done from the fixed head of the machine.

The weighing device, which is peculiar to the Emery machines, consists of a specially constructed hydraulic support in the fixed head of the machine, which is connected with a smaller hydraulic chamber in the scale case. The load on the test specimen is transmitted directly to the oil in the hydraulic support, and by hydrostatic pressure is transmitted to the hydraulic chamber which actuates the levers

* *Transactions*, Am. Soc. C. E., Vol. LXVI, p. 401.

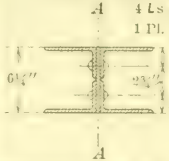
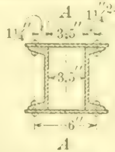
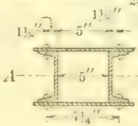
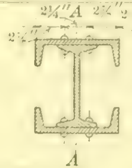
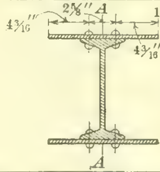
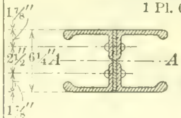
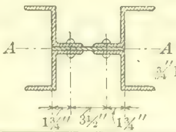
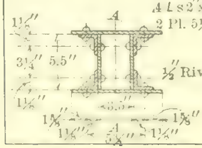
Light Section.	Area, in Square Inches.	r	$\frac{l}{r}$	Length.	Heavy Section.	Area, in Square Inches.	r	$\frac{l}{r}$	Length.
 4 Ls 5″ x 3″ x 5/16″ 1 Pl. 6″ x 5/16″ 5/8″ Rivets	9.60 1.875 11.475	2.24	120 85 50	22′4 13/16″ 15′10 5/16″ 9′4″	4 Ls 5″ x 3″ x 5/8″ 1 Pl. 6″ x 5/8″ A——Axis——A 5/8″ Rivets	18.44 3.75 22.19	2.36	120 85 50	23′7 3/16″ 16′8 5/8″ 9′10″
 2-6″ [s x 10.5-lb. 2 Pl. 8″ x 1/4″ 1/2″ Rivets	6.18 4.00 10.18	2.31	120 85 50	23′1 3/16″ 16′4 3/8″ 9′7 1/2″	2-6″ [s x 15.5-lb. 2 Pl. 8″ x 1/2″ 1/2″ Rivets	9.12 8.00 17.12	2.33	120 85 50	23′3 3/8″ 16′6 1/16″ 9′8 1/2″
 2-5″ [s x 6.5-lb. 2 Pl. 9 1/2″ x 1/4″ 1/2″ Rivets	3.90 4.75 8.65	2.34	120 85 50	23′4 15/16″ 16′6 3/4″ 9′9″	2 [s 5″ x 11.5-lb. 2 Pl. 9 1/2″ x 1/2″ 1/2″ Rivets	6.76 9.50 16.26	2.39	120 85 50	23′10 13/16″ 16′11 1/8″ 9′11 1/2″
 2 [s 8″ x 11 1/4-lb. 1 I 8″ x 18.0-lb. 5/8″ Rivets	6.70 5.33 12.03	2.38	120 85 50	23′9 5/8″ 16′10 5/16″ 9′11″	2 [s 8″ x 18.75-lb. 1 I 8″ x 20.50-lb. 5/8″ Rivets	11.02 6.03 17.05	2.32	120 85 50	23′2 1/16″ 16′5 3/16″ 9′8″

FIG. 1.

Light Section.	Area, in Square Inches.	r	$\frac{l}{r}$	Length.	Heavy Section.	Area, in Square Inches.	r	$\frac{l}{r}$	Length.
1 I 10″ x 25-lb. 2 Pl. 11″ x 5/16″  5/8″ Rivets	7.37 6.88 14.25	2.31	120 85 50	23′1 3/16″ 16′4 3/8″ 9′7 1/2″	1 I 10″ x 35-lb. 2 Pl. 11″ x 5/8″ 5/8″ Rivets	10.29 13.75 21.04	2.47	120 85 50	24′8 3/8″ 17′5 15/16″ 10′3 1/2″
4-5″ Bulb ∟s x 10.1-lb. 1 Pl. 6″ x 5/16″  5/8″ Rivets	11.88 1.88 13.76	2.48	120 85 50	24′9 5/8″ 17′6 13/16″ 10′4″	4-5″ Bulb ∟s x 13.5-lb. 1 Pl. 6″ x 7/16″ 5/8″ Rivets	15.88 2.63 18.51	2.42	120 85 50	24′2 3/8″ 17′1 11/16″ 10′1″
4-4″ Z's x 1/4″ 1 Pl. 7″ x 1/4″  3/4″ Rivets	9.64 1.75 11.39	2.46	120 85 50	24′7 3/16″ 17′5 1/8″ 10′3″	4-4″ Z's x 5/8″ 1 Pl. 7″ x 5/8″ 3/4″ Rivets	22.20 4.38 26.58	2.51	120 85 50	25′1 3/16″ 17′9 3/8″ 10′5 1/2″
2 Pl. 9″ x 1/4″ 4 ∟s 2″ x 2″ x 1/4″ 2 Pl. 5 1/4″ x 1/4″  1/2″ Rivets	4.500 3.760 2.625 10.885	2.33	120 85 50	23′3 5/8″ 16′6 1/16″ 9′8 1/2″	2 Pl. 9″ x 7/16″ 4 ∟s 2″ x 2″ x 7/16″ 2 Pl. 5 1/4″ x 7/16″ 1/2″ Rivets	7.88 6.24 4.59 18.71	2.34	120 85 50	23′4 13/16″ 16′6 7/8″ 9′9″

General Notes:
Rivet Spacing = 4 diameters of rivet
End distance = 2 " "
Make three columns of each length for all shapes

FIG. 2.

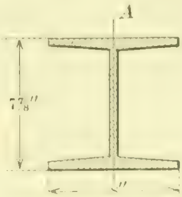
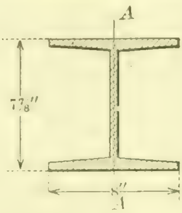
Column and section.		Area □″	r	$\frac{l}{r}$	Length. Ft.	Length. Ins.
Light Section. { Bethlehem. 8-in. H-Column. 8 in. by 31.5 lb.		9.17 9.17 9.17	1.98 1.98 1.98	120 85 50	19 14 8	9⅝ 0 5/16 3
Heavy Section. { Bethlehem. 8-in. H-Column. 8 in. by 62 lb.		18.27 18.27 18.27	2.09 2.09 2.09	120 85 50	20 14 8	10 13/16 9⅝ 8½

Fig. 3.

of the weighing scale. These levers are all of the plate-fulcra type, characteristic of all Emery testing machines.

The construction of the hydraulic support and chamber is such that a movement of 0.01 in. of the brass diaphragms is sufficient to register the full capacity of the machine. By use of these diaphragms and guiding rings, friction of the parts is eliminated.

The programme calls for all the tests to be made with square-ended columns, and this is done by providing cast-steel bearing blocks at each head, which bear directly against the columns. Considerable difficulty has been found in securing bearing blocks of sufficient accuracy, and it was necessary for the Bureau to go over all the blocks and dress them to a surface with a variation of not more than plus or minus 0.001 in.

For measuring the compressions on the longitudinal axis of the columns, compressometers were used, which consisted of a bar, fixed at one end to the column, and the other end connected with a micrometer gauge, which reads by estimate to ten-thousandths of an inch. This gauge was equipped with an electrical buzzer, which enabled the operator always to secure an equal contact. At the beginning of the series, wood bars were used, but, owing to the length of time taken in making a test, it was decided to substitute hollow steel bars, in order to avoid any possibility of changes due to moisture and temperature.

The local stresses were measured with a Berry strain gauge, which is made with a pair of Invar steel bars some 10 in. long, with a fixed point at one end and a movable point with a bell-crank at the other end. The horizontal arm is connected with an Ames dial gauge, which reads, with the multiplication of the bell crank, to 0.0002 in., and by estimating to 0.00002 in. The nominal length between the contact points of this strain gauge is 8 in.

The strain gauge measurement was made in the usual way, *viz.*, two holes were drilled, 8 in. apart, using a No. 55 drill, and then countersunk with a tool having an angle of about 110 degrees. This insures a good shoulder for the points on the strain gauge, which is highly essential for accurate work with this type of instrument. Experienced operators can secure very uniform readings within limits of plus or minus 0.0002 in.

Originally, the horizontal and vertical deflections of the column under test were measured with a piano wire, with a spring tension which was fastened to plugs set in each end of the column. Measurements were taken at the center by scales. It was found that when the ends of the column began to deflect, or warp, the local bends were transmitted to the wire, and this method was then improved by stretching the wire from a fixed frame below the column. The difficulty in securing accurate measurements from the wire to the column within limits of plus or minus 0.01 in. still remained, and it was finally overcome by abandoning the longitudinal wire entirely, and connecting the column at the center with two dial gauges reading to 0.001 in., which were supported on frames, independent of the column. Tension wires perpendicular to the longitudinal axis of the column transmitted the horizontal and vertical movements to the two gauges. Two operators were employed in manipulating the gauges, and, due to the length of time required to read these, some slight flow of metal may have taken place during the reading, but this was hardly sufficient to change the general result.

Test Columns.

The Progress Report of the Committee, of January 15th, 1913,* shows the column sections which the Committee proposed to use, together with the chemical and physical properties. The diagram of these sections and the properties are incorporated with the present report. In addition, it should be noted from this report that the programme was "arranged with the intention, as far as practicable, of eliminating all variables except that of form", and was subdivided so as to consider the effect of light and heavy sections. Three columns of each slenderness ratio, 50, 85, and 120 of $\frac{l}{r}$, were provided for

* *Proceedings,* Am. Soc. C. E., for February, 1913.

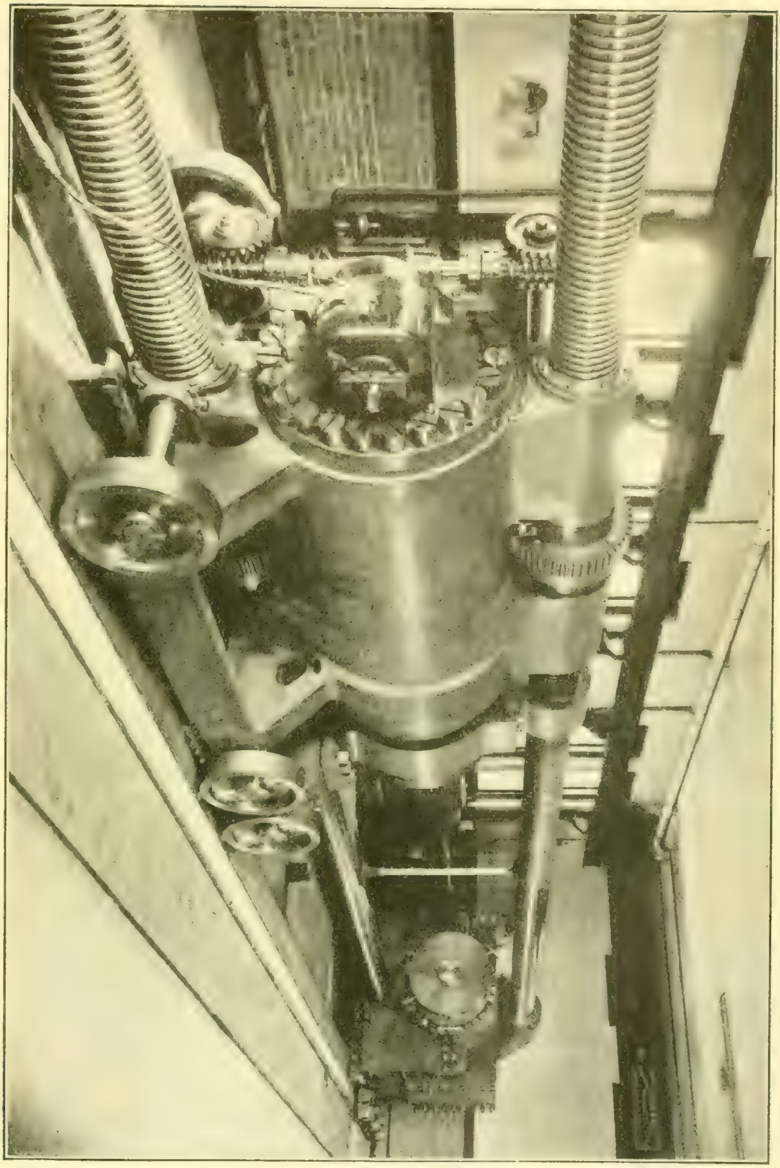


Fig. 4.—2 300 000-Pound Testing Machine, Bureau of Standards, Washington, D. C.

all shapes, making nine light sections and nine heavy sections, for each particular form of column, or a total of 144 test columns.

In addition to these, the Bureau of Standards has provided eighteen columns of Bethlehem **H**-sections, and eighteen columns of Carnegie 6-in. and 8-in. **H**-sections.

TABLE OF CHEMICAL AND PHYSICAL PROPERTIES FOR SPECIAL SPECIFICATIONS FOR UNIFORM QUALITY STRUCTURAL STEEL FOR COMPARATIVE COLUMN TESTS FOR THE BUREAU OF STANDARDS, U. S. GOVERNMENT.

Material.—Basic open-hearth structural steel.

Chemical.—Uniformity is paramount, and must be aimed for.

Carbon.—The percentage of carbon must not vary more than just to insure the desired physical properties in required sections of material.

Manganese	0.40% min. and 0.50% max.
Phosphorus	0.00% min. and 0.03% max.
Sulphur	0.00% min. and 0.04% max.
Silicon	0.00% min. and 0.10% max.
Nickel	0.00% min. and 0.05% max.
Chromium	0.00% min. and 0.05% max.
Copper	0.00% min. and 0.03% max.
All possible others	0.00%

Physical.—The desired physical properties must be aimed for.

Ultimate tensile strength	Desired, 60 000 lb. per sq. in. (may vary 1 500 lb. either way).
Elastic limit (yield point)	Desired, 38 000 lb. per sq. in. (may vary 1 000 lb. either way).
Elongation in 8 in.	Desired, 28% (may vary 2% either way).
Reduction in area	Desired, 56% (may vary 4% either way).
Character of fracture	Silky, cup.
Cold bend without fracture	180°, flat, on itself.

During the rolling of the material at the mill, and the fabrication at the shop, the inspectors of the Bureau of Standards investigated the various processes, making notes of the different conditions to which the material was subjected. Care was taken that the material met the rigid specifications, and although some variations were allowed, it was only with the special permission of the Committee.

In addition to the material required for building the columns, the Bureau of Standards ordered additional pieces, 5 ft. long, of each shape and from each melt, so as to have an extra lot of unworked material on hand for special physical and chemical tests. The physical and

chemical tests were first made on all shapes by the manufacturer, and were then checked at the laboratory of the Bureau of Standards, at Pittsburgh.

It will be noted from the diagrams (Figs. 1, 2, and 3) showing the cross-sections of the columns that the rivets were all of standard size, the smallest being ½ in. and the largest ¾ in. in diameter, and the rivet spacing called for four diameters throughout the total length of the columns. It was the intention of the Committee to over-rivet the sections, so as to insure the parts acting together as a whole. So far, in no case has a failure occurred due to a bulging of the material between rivets. Where the specimens have been compressed, in order to emphasize the method of failure, the rivets in all cases have remained tight.

In general, the shop work may be described as being uniformly good, and better than the average commercial grade. The ends were dressed square to the longitudinal axis of the column, but in almost all cases it was found necessary to re-dress them, owing to the fact that the ordinary commercial milling machine gives too rough a cut to insure complete bearing over the whole surface of the end cross-section of the column. After the test column was placed in the machine, special attention was given to insure a uniform bearing by applying loads of about 1 000 lb. per sq. in.

The investigation of the test columns developed variations in their sectional areas, these amounting, in extreme cases, to as much as 4% from the nominal area. In order to determine the correct area, the shapes were first calipered at the ends of the columns, and the variations noted. Prints of both ends were taken by blacking the ends and pressing white paper against them. The area was then determined by running a planimeter around the inside and outside perimeters. Finally, the column was weighed, the weight of the rivet heads was deducted, and the area was determined by the specific gravity of the material. This area, after being compared and checked with the results of the other two methods, was adopted as the correct one to be used in determining the unit loadings. When the column was put in the machine, before being tested, it was carefully examined for loose rivets, scale, or any other conditions which might affect its strength, and, during progress of the tests, rivets were tried and any scaling was noted. After the column was placed in the machine, a nominal load, of sufficient amount to insure its being held to the proper bearings, was applied, and the compressometers were attached. For the $\frac{l}{r}$ ratio of 50, 85, and 120, the test pieces ran on an average of 10, 16, and 23 ft. long, and the compressometers were arranged respectively with bars 50, 100, and 150 in. long. The two tests given*

* The details of these tests will be given later.

will indicate the details of applying the loads, and, as stated before, after each increment of 1 000 lb. per sq. in. had been applied, sufficient time was given to read the various strain gauges.

Failure of the column was first indicated by the dial gauges, which registered a rapid increase of the deflection at the center, and by the unbalancing of the scale beam, which showed a falling off of resistance to compression. With the exception of some additional scaling, the column at this time showed no apparent change, and the pressure could be maintained for several minutes before it had a perceptible bend.

As soon as the maximum loading was reached, the instruments were taken off, and one column of each series of three was further compressed, so as to emphasize the failure, and a photograph was taken to indicate how the column failed.

Methods of Testing.

After the material was ready for the beginning of the programme of testing, a conference was held between the physicists of the Bureau of Standards and a Sub-Committee of the Column Committee as to the methods of applying the loads and measuring the strains. A general outline of the instructions to the Engineer in Charge was agreed upon, and this outline is given below. Thus far it has not been necessary to make any marked changes in the methods. On certain tests, variations have been introduced, at the request of the Committee, or at the suggestion of the physicist, but, thus far, these have not produced enough variation from the general result to warrant their continuance.

Outline of Instructions for Testing.—Columns shall be weighed and gauged for sizes before being tested. In addition, inspection shall be made to determine the condition of the riveting, in order to see that the pieces are properly drawn together, especial note being made of any local conditions where the riveting has failed to do this. Observations shall also be recorded where the pieces at the ends of the members have been bent. General alignment, waves in pieces or in the column as a whole, kinks, or any other special condition shall be noted.

The ends of the columns shall be dressed flat to reduce the amount of shimming to a minimum.

The two longer series of columns, that is, those of 85 and 120 $\frac{l}{r}$, shall be counterweighted at the middle by an amount equal to one-half of the total weight of the column.

Strain gauge measurements along the length of the column, at points from 8 to 10 in. apart, shall be made at the center of the column and near both ends. The gauge points at the ends shall be

placed far enough away to avoid effects of local distortion due to the taking up of the stresses. These strain gauge measurements shall be made at each of the four corners of the column and at the center of the webs and flanges. Special shapes, such as the Z-bar and bulb angle shape columns, may require variations from this general rule in order to determine the variation in stress in the different parts of the column.

Compressometer measurements shall also be made on longer lengths, of from 50 to 150 in., in order to check the short strain gauge measurements. Four of these will be made on each column—two on the corners and one on the center of the flange, and one on the center of the web.

Deflection observations shall be made on the horizontal and vertical movements at the middle of the length of each test piece.

The condition of the cross-section of the column shall be determined by calipering the two open sides before applying the initial load, and at intervals of loading of 5 000 lb. per sq. in., in order to determine any change in the shape of the cross-section. This shall be done in as many places as may be necessary.

Initial loads of 1 000 lb. per sq. in. each shall be applied, and compression loads of 5 000, 10 000, and 15 000 lb. per sq. in. each, returning to the initial load and determining permanent sets; from 15 000 lb. proceeding with increments of 1 000 lb. per sq. in. each, determining permanent sets after loads of 20 000 and 25 000 lb. per sq. in.; also at 30 000 lb. per sq. in., if these several loads are reached before the column fails.

Horizontal and vertical deflections of the columns shall be observed at the time the longitudinal compressions and sets are measured. The general direction taken by the column when it fails, and whether due to deflection as a whole or local buckling, shall be noted.

Careful observations shall be made where the column first begins to scale, also the lines of scale. After failure, record the conditions of the rivets around the distortion points to see if any of these rivets are loose.

After the ultimate load has been reached, at least one column of each cross-section and length shall be compressed, to emphasize the manner of failure.

In general, photographs shall be taken of one column of each cross-section and length when the ultimate load has been reached, and in addition a photograph of each column on which the failure has been emphasized.

Examples of Tests.

The Committee gives the record of two of the tests, *Nos. 30 and 54, which contain the information as furnished by the Bureau of Standards.

* Plates LVI and LVII.

Column Test Record
Form 260 a.

DEPARTMENT OF COMMERCE, BUREAU OF STANDARDS, WASHINGTON, D. C., REPORT ON TEST No. 30

Date, April 3 and 4, 1916.

For whom tested : American Society of Civil Engineers.
Type No. or designation : 1.
Marks on column : 1 A C 6451.
Tested with flat ends.

Counterweighted : one-half its weight at middle.
Nominal sectional area, in square inches : 11.475.
Radius of gyration : 2.24.
Slenderness ratio : 120.

Initial condition—
Riveting : Good.
Members at the ends : Good.
Alignment : Good (See note).

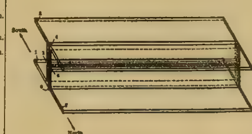
Length over all : 22 ft. 4 1/2 in.
Weight in pounds : 898.
Sectional area, in square inches : 11.36. Corrected area, 11.20.
Gauged lengths : 150 and 8 in.

Applied Loads. Pounds, total.	Applied Loads. *Pounds per square inch.	Compressions, in Inches, in 150-in. Gauged Lengths for Positions. 1	2	3	4	5	Mean compression in 150-in. gauged lengths (inches).	Deflections at Middle of Length (Inches). Horizontal. North.	Vertical. Down.
11 360	1 000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	....	......
56 800	5 000	0.0214	0.0212	0.0196	0.0205	0.0211	0.0208	0.00	0.00
11 360	1 000	0.0002	0.0005	0.0009	0.0004	0.0001	0.0001	....	......
56 800	5 000	0.0214	0.0212	0.0196	0.0206	0.0211	0.0208		
113 600	10 000	0.0470	0.0465	0.0459	0.0471	0.0461	0.0465	0.00	0.00
11 360	1 000	0.0020	0.0021	0.0004	0.0024	0.0004	0.0011	....	......
56 800	5 000	0.0224	0.0215	0.0191	0.0225	0.0186	0.0208		
113 600	10 000	0.0469	0.0475	0.0449	0.0476	0.0446	0.0463		
170 400	15 000	0.0722	0.0742	0.0716	0.0747	0.0720	0.0729	0.00	0.00
11 360	1 000	0.0022	0.0027	0.0015	0.0034	0.0011	0.0011	....	......
11 360	1 000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	....	......
56 800	5 000	0.0210	0.0206	0.0210	0.0210	0.0212	0.0210		
113 600	10 000	0.0463	0.0466	0.0473	0.0473	0.0471	0.0468		
170 400	15 000	0.0717	0.0723	0.0734	0.0732	0.0739	0.0729		
181 760	16 000	0.0766	0.0783	0.0789	0.0787	0.0796	0.0784	0.00	0.00
193 120	17 000	0.0812	0.0828	0.0841	0.0840	0.0846	0.0833	0.00	0.005 D
204 480	18 000	0.0860	0.0880	0.0892	0.0897	0.0901	0.0886	0.00	0.01
215 840	19 000	0.0911	0.0934	0.0949	0.0956	0.0960	0.0942	0.00	0.01
227 200	20 000	0.0953	0.1008	0.1007	0.1014	0.1021	0.1001	0.00	0.02
11 360	1 000	0.0008	0.0015	0.0010	0.0025	0.0012	0.0013	0.00	0.005
56 800	5 000	0.0205	0.0212	0.0209	0.0223	0.0210	0.0212		
113 600	10 000	0.0460	0.0473	0.0469	0.0486	0.0473	0.0472		
170 400	15 000	0.0712	0.0785	0.0784	0.0750	0.0744	0.0735		
227 200	20 000	0.0963	0.0999	0.1006	0.1016	0.1017	0.1000		
238 560	21 000	0.1013	0.1066	0.1059	0.1077	0.1075	0.1056	0.00	0.02
249 920	22 000	0.1058	0.1121	0.1122	0.1141	0.1143	0.1117	0.00	0.02
261 280	23 000	0.1106	0.1184	0.1169	0.1212	0.1210	0.1180	0.00	0.03
272 640	24 000	0.1151	0.1254	0.1267	0.1285	0.1293	0.1250	0.00	0.04
284 000	25 000	0.1188	0.1315	0.1354	0.1369	0.1368	0.1319	0.00	0.06
11 360	1 000	0.0012	0.0057	0.0079	0.0095	0.0076	0.0059	0.00	0.05
56 800	5 000	0.0197	0.0273	0.0288	0.0299	0.0293	0.0270		
113 600	10 000	0.0448	0.0628	0.0550	0.0568	0.0558	0.0530		
170 400	15 000	0.0696	0.0793	0.0819	0.0829	0.0830	0.0793		
227 200	20 000	0.0944	0.1055	0.1089	0.1106	0.1121	0.1068		
284 000	25 000	0.1161	0.1338	0.1374	0.1361	0.1398	0.1330		
318 700	28 066	......	......	......	......	......	......	0.30 N	1.00 D
318 700	28 456	Ultimate Strength According to Corrected Area.						....	......

Pounds, total.	40 in. from Base. Compression, in Inches, in 8-in. Gauged Lengths at Base for Positions. 1	2	3	4	5	6	7	8	Straddling Middle. Compression, in Inches, in 8-in. Gauged Lengths at Middle for Positions. 1	2	3	4	5	6	7	8	Remarks.
11 360	0.0000	......	0.0000	0.0000	0.0000	......	0.0000	0.0000	0.0000	......	0.0000	......	0.0000	......	0.0000	0.0000	Temp. 20.9° cent.
56 800	0.0013	......	0.0018	0.0010	0.0011	......	0.0009	0.0011	0.0013	......	0.0012	......	0.0014	......	0.0012	0.0014	Note: 42 in. from top of column, bad kink in leg of angle along line of gaugings.
11 360	0.0001	......	0.0001	0.0004	0.0001	......	0.0001	0.0000	0.0001	......	0.0000	......	0.0000	......	0.0001	0.0002	No. 1 leg of angle bent in 0.6 in. The behavior of the angle is shown below.
56 800																	
113 600	0.0027	......	0.0027	0.0023	0.0026	......	0.0025	0.0025	0.0026	......	0.0026	......	0.0025	......	0.0028	0.0029	
11 360	0.0001	......	0.0000	0.0004	0.0002	......	0.0001	0.0001	0.0001	......	0.0000	......	0.0000	......	0.0002	0.0004	
56 800																	
113 600																	
170 400	0.0042	......	0.0043	0.0037	0.0040	......	0.0040	0.0081	0.0041	......	0.0041	......	0.0043	......	0.0043	0.0044	Temp. 21° cent.
11 360	0.0001	......	0.0000	0.0004	0.0002	......	0.0001	0.0001	0.0001	......	0.0000	......	0.0003	......	0.0004	0.0004	
11 360	0.0000	......	0.0000	0.0000	0.0000	......	0.0000	0.0000	0.0000	......	0.0000	......	0.0000	......	0.0000	0.0000	April 4th. Temp. 19.5° cent.
56 800																	
113 600																	
170 400																	
181 760	0.0044	......	0.0046	0.0044	0.0043	......	0.0044	0.0043	0.0044	......	0.0044	......	0.0046	......	0.0044	0.0046	Snapping noise.
193 120	0.0047	......	0.0048	0.0048	0.0045	......	0.0047	0.0046	0.0047	......	0.0049	......	0.0049	......	0.0046	0.0049	
204 480	0.0050	......	0.0052	0.0052	0.0047	......	0.0050	0.0049	0.0050	......	0.0052	......	0.0051	......	0.0048	0.0052	
215 840	0.0053	......	0.0055	0.0056	0.0051	......	0.0054	0.0052	0.0052	......	0.0056	......	0.0055	......	0.0051	0.0055	Snapping noise.
227 200	0.0055	......	0.0058	0.0060	0.0054	......	0.0058	0.0064	0.0054	......	0.0059	......	0.0058	......	0.0056	0.0059	Slight drop of beam.
11 360	0.0001	......	0.0001	0.0003	0.0001	......	0.0000	0.0000	0.0000	......	0.0002	......	0.0000	......	0.0001	0.0008	Slight loosening of scale on angles.
56 800																	
113 600																	
170 400																	
227 200																	
238 560	0.0059	......	0.0061	0.0063	0.0058	......	0.0061	0.0059	0.0058	......	0.0063	......	0.0061	......	0.0059	0.0062	
249 920	0.0062	......	0.0065	0.0068	0.0061	......	0.0064	0.0061	0.0061	......	0.0068	......	0.0066	......	0.0061	0.0066	
261 280	0.0065	......	0.0070	0.0072	0.0064	......	0.0068	0.0065	0.0064	......	0.0072	......	0.0070	......	0.0066	0.0069	Snapping noise.
272 640	0.0067	......	0.0072	0.0076	0.0068	......	0.0072	0.0070	0.0066	......	0.0078	......	0.0074	......	0.0068	0.0072	
284 000	0.0071	......	0.0077	0.0080	0.0070	......	0.0076	0.0074	0.0068	......	0.0086	......	0.0078	......	0.0071	0.0075	Legs of angles bending.
11 360	0.0001	......	0.0006	0.0010	0.0001	......	0.0002	0.0002	0.0000	......	0.0011	......	0.0002	......	0.0008	0.0008	Scaling pretty general on 5-in. legs of angles.
56 800																	
113 600																	
170 400																	
227 200																	
284 000																	
318 700	Failed primarily by buckling of legs of angles, but finally by buckling in center. Temperature at finish 20.5° cent.																
318 700																	

Applied Loads. Pounds, total.	*Pounds per square inch.	40 in. from Top. Compression, in Inches, in 8-in. Gauged Lengths at Top for Positions. 1	2	3	4	5	6	7	8	Compression, in Inches, in ......Gauged Lengths. 1	2	3	4	5	6	7	8
11 360	1 000	0.0000	......	0.0000	0.0000	0.0000	......	0.0000	0.0000	......	......	......	......	......	......	......	......
56 800	5 000	0.0011	......	0.0009	0.0008	0.0012	......	0.0012	0.0012	......	......	......	......	......	......	......	......
11 360	1 000	0.0000	......	0.0001	0.0001	0.0000	......	0.0001	0.0002	......	......	......	......	......	......	......	......
56 800	5 000	......	......	......	......	......	......	......	......	......	......	......	......	......	......	......	......
113 600	10 000	0.0024	......	0.0022	0.0023	0.0020	......	0.0026	0.0030	......	......	......	......	......	......	......	......
11 360	1 000	0.0001	......	0.0001	0.0001	0.0000	......	0.0001	0.0000	......	......	......	......	......	......	......	......
56 800	5 000	......	......	......	......	......	......	......	......	......	......	......	......	......	......	......	......
113 600	10 000	......	......	......	......	......	......	......	......	......	......	......	......	......	......	......	......
170 400	15 000	0.0035	......	0.0040	0.0036	0.0038	......	0.0037	0.0050	......	......	......	......	......	......	......	......
11 360	1 000	0.0002	......	0.0000	0.0002	0.0000	......	0.0001	0.0001	......	......	......	......	......	......	......	......
11 360	1 000	0.0000	......	0.0000	0.0000	0.0000	......	0.0000	0.0000	......	......	......	......	......	......	......	......
56 800	5 000	......	......	......	......	......	......	......	......	......	......	......	......	......	......	......	......
113 600	10 000	......	......	......	......	......	......	......	......	......	......	......	......	......	......	......	......
170 400	15 000	......	......	......	......	......	......	......	......	......	......	......	......	......	......	......	......
181 760	16 000	0.0042	......	0.0043	0.0042	0.0040	......	0.0041	0.0063	......	......	......	......	......	......	......	......
193 120	17 000	0.0044	......	0.0046	0.0044	0.0044	......	0.0043	0.0064	......	......	......	......	......	......	......	......
204 480	18 000	0.0047	......	0.0050	0.0046	0.0047	......	0.0045	0.0071	......	......	......	......	......	......	......	......
215 840	19 000	0.0050	......	0.0054	0.0048	0.0049	......	0.0046	0.0076	......	......	......	......	......	......	......	......
227 200	20 000	0.0053	......	0.0058	0.0052	0.0060	......	0.0049	0.0082	......	......	......	......	......	......	......	......
11 360	1 000	0.0000	......	0.0004	0.0000	0.0000	......	0.0004	0.0019	......	......	......	......	......	......	......	......
56 800	5 000	......	......	......	......	......	......	......	......	......	......	......	......	......	......	......	......
113 600	10 000	......	......	......	......	......	......	......	......	......	......	......	......	......	......	......	......
170 400	15 000	......	......	......	......	......	......	......	......	......	......	......	......	......	......	......	......
227 200	20 000	......	......	......	......	......	......	......	......	......	......	......	......	......	......	......	......
238 560	21 000	0.0056	......	0.0062	0.0056	0.0053	......	0.0049	0.0087	......	......	......	......	......	......	......	......
249 920	22 000	0.0069	......	0.0068	0.0057	0.0055	......	0.0049	0.0094	......	......	......	......	......	......	......	......
261 280	23 000	0.0062	......	0.0072	0.0060	0.0067	......	0.0051	0.0103	......	......	......	......	......	......	......	......
272 640	24 000	0.0064	......	0.0077	0.0062	0.0058	......	0.0049	0.0113	......	......	......	......	......	......	......	......
284 000	25 000	0.0068	......	0.0082	0.0064	0.0053	......	0.0046	0.0124	......	......	......	......	......	......	......	......
11 360	1 000	0.0001	......	0.0011	0.0004	0.0004	......	0.0017	0.0044	......	......	......	......	......	......	......	......

Pounds, total.	Variations in Distances Between Legs of Angles, at Bend in Flange. Start of bend.	Max. bend or center.	Start of bend.	South side. 1/4	South side. 1/2	South side. 3/4	North side. 1/4	North side. 1/2	North side. 3/4
11 360	5.592	5.015	5.581						
56 800	5.591	5.008	5.585	0.000	......	0.000	0.000	......	0.002 open.
11 360	5.589	5.015	5.581						
56 800	......	......	......						
113 600	5.600	4.998	5.588	......	......	......	......	......	0.004 open.
11 360	5.593	4.018	5.581						
56 800	5.606	4.985	5.594	......	......	......	......	......	0.006 open.
113 600	......	......	......						
170 400	5.594	4.011	5.588						
11 360	......	......	......						
11 360	......	......	......						
56 800	......	......	......						
113 600	......	......	......						
170 400	......	......	......						
181 760	5.606	4.974	5.596						
193 120	5.607	4.971	5.596						
204 480	......	......							
215 840	......	......							
227 200	......	......							
11 360	......	5.24							
56 800	......	......							
113 600	......	......							
170 400	......	......							
227 200	......	5.19	......	......	......	......	......	......	0.01 open.
238 560	......	5.18							
249 920	......	5.17							
261 280	......	5.16							
272 640	......	5.14							
284 000	......	5.20							
11 360	......	5.01							



Test No. 30.

* Sectional area used in computing unit loading, 11.36 sq. in. Corrected area, 11.20 sq. in. All loadings in pounds per square inch should be adjusted by multiplying by the factor, 1.0143.

Details of Test No. 30, to Accompany Progress Report of Special Committee on Steel Columns and Struts.

TABLE 1.

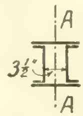
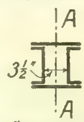
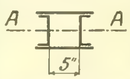
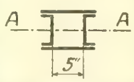
ABSTRACT OF RECORD OF COLUMN TESTS.

TYPES I & IA – I & IA SPECIAL.

Type	Test no.	Section.	Actual Area	Radius of Gyration	Length	$\frac{l}{r}$	Unit Ultimate Strength.	Average Ultimate Strength.	Deflection in inches at the Middle of Columns at the Reading just before Failure.	
	8	Light Sections	11.13	2.24	9'–4"	50	32400		0.035 S.	0.000
	38		11.17	"	"	"	32800	32700	0.07 S.	0.1000 Up.
	77		11.13	"	"	"	33000		0.050 N.	0.0313 Up.
	18		11.49	"	15'–10 7/16"	85	32100		0.12 S.	0.05 Up.
I.	84		11.49	"	"	"	31400	31200	0.0252 S.	0.1281 Down.
	85	4 L^s 5"x3"x 5/16" = 9.60 □"	11.35	"	"	"	30000		0.0626 N.	0.1368 Up.
		1 Pl. 6"x 5/16" = 1.875								
	30	Total = 11.475 □"	11.20	"	22'–4 13/16"	120	28600		0.000	0.060 Down.
	56		11.14	"	"	"	27600	28300	0.1175 S.	0.1091 Up.
	69		11.30	"	"	"	28700		0.0135 N.	0.1785 Down.
	93	Heavy Sections.	22.25	2.36	9'–10"	50	29000		0.0080 S.	0.0574 Down.
	96		22.13	"	"	"	28900	29200	0.0731 N.	0.0457 Up.
	97		22.28	"	"	"	29700		0.0653 N.	0.0002 Up.
										
	91		22.134	"	16'–8 5/8"	85	28000		0.0475 S.	0.0883 Up.
IA	94		21.92	"	"	"	28200	28100	0.4053 N.	0.2636 Up.
	95	4 L^s 5"x3"x 5/8" = 18.44 □"	22.05	"	"	"	28000		0.0764 N.	0.0825 Down.
		1 Pl. 6"x 5/8" = 3.75								
	92	Total = 22.19 □"	22.15	"	23'–7 3/16"	120	25000		0.1430 S.	0.2923 Down.
	98		22.10	"	"	"	25500	25400	0.3863 N.	0.0762 Up.
	99		22.10	"	"	"	25800		0.1326 N.	0.0606 Down.
	85A	Light Sections	11.35	2.24	3'–8 3/4"	20	35100		0.0089 N.	0.0600 Down.
	84A		11.49	"	"	"	38000	36400	0.0230 S.	0.0313 Up.
	84B		11.49	"	"	"	36100		0.0141 S.	0.0120 Down.
I Spec.		as above.								
	99A	Heavy Sections	22.10	2.36	3'–11 3/16"	20	45000		0.0209 S.	0.0157 Down.
	98A		"	"	"	"	44100	44400	0.0177 N.	0.0104 Down.
	98B		"	"	"	"	44100		0.0031 N.	0.0229 Up.
IA Spec.		as above.								

TABLE 2.

ABSTRACT OF RECORD OF COLUMN TESTS.
TYPES 2 & 2A – 3 & 3A

Type	Test no.	Section.	Actual Area	Radius of Gyration	Length	$\frac{l}{r}$	Unit Ultimate Strength	Average Ultimate Strength	Deflection in inches at the Middle of Columns at the Reading just before Failure	
	11	Light Sections.	10.06	2.31	9'-7½"	50	33100		0.08 S.	0.00
	41		10.15	"	"	"	34000	33200	0.0024 N.	0.0374 Up.
	43		10.06	"	"	"	32400		0.0460 S.	0.0441 Up.
	21		9.92	"	16'-4⅜"	85	33700		0.12 S.	0.04 Up.
2	59		9.87	"	"	"	32300	32600	0.1195 N.	0.0195 Up.
	65	2-6" [s × 10.5-lb = 6.18 □"	10.05	"	"	"	31700		0.0627 N.	0.0927 Down
		2 Pls 8"×¼" = 4.00								
	32	Total = 10.18 □"	10.01	"	23'-1 3/16"	120	28300		0.27 S.	0.12 Up.
	54		10.09	"	"	"	28700	29300	0.2750 S.	0.0583 Up.
	70		10.07	"	"	"	30900		0.1725 N.	0.0144 Up.
	101	Heavy Sections.	17.07	2.33	9'-8½"	50	32200		0.0318 N.	0.0360 Up.
	124		17.07	"	"	"	32500	32300	0.0052 N.	0.0044 Up.
	165		16.96	"	"	"	32300		0.1905 S.	0.0131 Up.
	145		17.07	"	16'-6 1/16"	85	30700		0.0416 N.	0.0820 Down.
2A	146		17.02	"	"	"	30000	30600	0.0606 S.	0.0731 Up.
	151	2-6" [s × 15.5-lb = 9.12 □"	17.11	"	"	"	31000		0.0312 S.	0.0078 Down
		2 Pls 8"×½" = 8.00								
	135	Total = 17.12 □"	17.04	"	23'-3⅝"	120	28500		0.0668 S.	0.0298 Up.
	136		17.02	"	"	"	28800	28100	0.1315 S.	0.1942 Up.
	137		17.08	"	"	"	27000		0.1503 N.	0.0339 Down.
	1	Light Sections.	8.71	2.34	9'-9"	50	33800		0.01 N.	0.06 Up.
	2		8.80	"	"	"	34300	34100	0.01 S.	0.04 Up.
	42		8.71	"	"	"	34300		0.0418 N.	0.0306 Up.
	14		8.71	"	16'-6⅞"	85	32500		0.10 N.	0.06 Down.
3	87		8.69	"	"	"	31900	32400	0.0764 S.	0.1994 Down.
	88	2-5" [s × 6.5-lb = 3.90 □"	8.60	"	"	"	32900		0.0470 S.	0.1774 Up.
		2 Pls 9½"×¼" = 4.75								
	24	Total = 8.65 □"	8.79	"	23'-4 13/16"	120	30600		0.13 N.	0.01 Up.
	35		8.70	"	"	"	30200	30300	0.17 S.	0.08 Down.
	47		8.74	"	"	"	30000		0.2056 N.	0.1404 Down.
	105	Heavy Sections.	16.44	2.39	9'-11½"	50	29800		0.0084 N.	0.1044 Down.
	168		16.45	"	"	"	29300	29500	0.0077 N.	0.0170 Up.
	171		16.39	"	"	"	29500		0.0126 N.	0.0228 Up.
	147		16.41	"	16'-11⅛"	85	28000		0.0574 S.	0.0721 Down.
3A	148		16.39	"	"	"	28000	28000	0.0313 N.	0.0470 Down.
	150	2-5" [s × 11.5-lb = 6.76 □"	16.37	"	"	"	28000		0.1159 N.	0.0835 Up.
		2 Pls 9½"×½" = 9.50								
	138	Total = 16.26 □"	16.50	"	23'-10 13/16"	120	28000		0.1848 S.	0.0449 Down.
	139		16.45	"	"	"	26100	26900	0.0919 N.	0.3284 Down.
	143		16.42	"	"	"	26600		0.1201 N.	0.3920 Up.

COLUMN TEST RECORD
Form 260 a.

DEPARTMENT OF COMMERCE, BUREAU OF STANDARDS, WASHINGTON, D. C.—REPORT ON TEST No. 54.

Date, July 24, 1914.

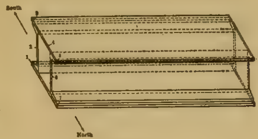
For whom tested: American Society of Civil Engineers.
Type No. or designation: 2.
Marks on column: 2A.
Tested with flat ends.

Counterweighted half its weight at middle.
Nominal sectional area, in square inches: 10.15.
Radius of gyration: 2.31.
Slenderness ratio: 120.

Initial condition—
Riveting: Good.
Members at the ends: Good.
Alignment: Vertical, good. Bent south 0.2 in. in center.

Length over all: 23 ft. $1\frac{3}{16}$ in.
Weight in pounds: 840.
Sectional area, in square inches: 9.66.
Gauged lengths: 150 and 8 in. Corrected area, 10.09 sq. in.

Applied Loads.		Compressions, in Inches, in 150-in. Gauged Lengths for Positions.					Mean compression in 150-in. gauged lengths (inches).	Deflections at Middle of Length (Inches).		Straddling Middle. Compression, in Inches, in 8-in. Gauged Lengths at Middle for Positions.								Remarks.
Pounds, total.	*Pounds per square inch.	1	3	5	7	4		Horizontal. South.	Vertical. Up.	1	2	3	4	5	6	7	8	
9 660	1 000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	......	0.0000	0.0000	0.0000	0.0000	
48 300	5 000	0.0197	0.0183	0.0204	0.0210	0.0198	0.0198	0.0097	0.0097	0.0012	0.0011	0.0010	......	0.0012	0.0011	0.0011	0.0011	
9 660	1 000	0.0006	0.0002	0.0001	0.0002	0.0003	0.0001	0.0005	0.0184	0.0000	0.0000	0.0000	......	0.0001	0.0000	0.0000	0.0001	
48 300	5 000	0.0198	0.0183	0.0202	0.0210	0.0212	0.0201											
96 600	10 000	0.0440	0.0420	0.0464	0.0477	0.0459	0.0452	0.0234	0.0172	0.0025	0.0024	0.0023	......	0.0026	0.0025	0.0027	0.0025	
9 660	1 000	0.0011	0.0003	0.0004	0.0000	0.0020	0.0008	0.0016	0.0143	0.0000	0.0001	0.0000	......	0.0001	0.0000	0.0000	0.0001	
48 300	5 000	0.0203	0.0184	0.0209	0.0265	0.0217	0.0216											
96 600	10 000	0.0443	0.0422	0.0469	0.0478	0.0463	0.0455											
144 900	15 000	0.0681	0.0656	0.0728	0.0745	0.0708	0.0704	0.0403	0.0214	0.0036	0.0035	0.0035	......	0.0041	0.0039	0.0041	0.0038	
9 660	1 000	0.0018	0.0005	0.0004	0.0014	0.0023	0.0012	0.0030	0.0156	0.0000	0.0002	0.0000	......	0.0002	0.0001	0.0002	0.0002	
48 300	5 000	0.0209	0.0193	0.0216	0.0226	0.0225	0.0214											
96 600	10 000	0.0446	0.0427	0.0472	0.0486	0.0469	0.0460											
144 900	15 000	0.0683	0.0657	0.0729	0.0747	0.0718	0.0706											
154 560	16 000	0.0728	0.0702	0.0784	0.0800	0.0763	0.0755	0.0444	0.0229	0.0040	0.0038	0.0037	......	0.0044	0.0042	0.0044	0.0042	
164 220	17 000	0.0778	0.0747	0.0836	0.0853	0.0810	0.0805	0.0484	0.0235	0.0043	0.0041	0.0039	......	0.0048	0.0046	0.0048	0.0045	Snapping noise.
173 880	18 000	0.0826	0.0793	0.0890	0.0906	0.0859	0.0855	0.0527	0.0246	0.0044	0.0044	0.0043	......	0.0051	0.0048	0.0050	0.0048	
183 540	19 000	0.0873	0.0836	0.0940	0.0962	0.0908	0.0904	0.0575	0.0259	0.0047	0.0046	0.0044	......	0.0053	0.0051	0.0054	0.0049	
193 200	20 000	0.0921	0.0883	0.0990	0.1020	0.0957	0.0958	0.0630	0.0272	0.0050	0.0050	0.0048	......	0.0057	0.0054	0.0057	0.0053	
9 660	1 000	0.0018	0.0005	0.0011	0.0023	0.0028	0.0017	0.0061	0.0174	0.0001	0.0002	0.0001	......	0.0001	0.0000	0.0001	0.0001	
48 300	5 000	0.0210	0.0192	0.0219	0.0233	0.0228	0.0216	0.0162	0.0152									
96 600	10 000	0.0450	0.0429	0.0481	0.0498	0.0473	0.0466	0.0297	0.0205									
144 900	15 000	0.0687	0.0653	0.0739	0.0755	0.0721	0.0711	0.0450	0.0242									
198 200	20 000	0.0923	0.0887	0.1000	0.1021	0.0962	0.0959	0.0637	0.0282									
193 200	20 000	0.0923	0.0889	0.1006	0.1022	0.0961	0.0960	......	......	......	......	......	......	......	......	......	......	After 5 min.
193 200	20 000	0.0924	0.0891	0.1001	0.1023	0.0961	0.0960	......	......	......	......	......	......	......	......	......	......	After 10 min.
193 200	20 000	0.0924	0.0892	0.1004	0.1022	0.0961	0.0061	0.0640	0.0282	......	......	......	......	......	......	......	......	After 15 min.
202 860	21 000	0.0970	0.0934	0.1055	0.1077	0.1010	0.1009	0.0684	0.0289	0.0052	0.0052	0.0050	......	0.0061	0.0058	0.0061	0.0056	
212 520	22 000	0.1015	0.0981	0.1111	0.1135	0.1062	0.1061	0.0752	0.0307	0.0056	0.0054	0.0052	......	0.0064	0.0061	0.0064	0.0059	
222 180	23 000	0.1064	0.1025	0.1169	0.1192	0.1112	0.1112	0.0817	0.0324	0.0057	0.0058	0.0054	......	0.0066	0.0064	0.0068	0.0063	Snapping noise
231 840	24 000	0.1118	0.1075	0.1231	0.1260	0.1168	0.1170	0.0905	0.0348	0.0059	0.0060	0.0056	......	0.0070	0.0069	0.0072	0.0067	" "
241 500	25 000	0.1169	0.1121	0.1294	0.1327	0.1220	0.1226	0.0998	0.0379	0.0061	0.0062	0.0058	......	0.0075	0.0072	0.0077	0.0071	
9 660	1 000	0.0039	0.0022	0.0039	0.0096	0.0049	0.0043	0.0156	0.0226	0.0001	0.0001	0.0001	......	0.0001	0.0002	0.0002	0.0001	
48 300	5 000	0.0230	0.0209	0.0249	0.0272	0.0245	0.0241	0.0268	0.0204									
96 600	10 000	0.0470	0.0441	0.0512	0.0540	0.0493	0.0491	0.0407	0.0257									
144 900	15 000	0.0709	0.0677	0.0769	0.0800	0.0740	0.0789	0.0575	0.0300									
193 200	20 000	0.0943	0.0900	0.1033	0.1072	0.0986	0.0987	0.0783	0.0344									
241 500	25 000	0.1175	0.1124	0.1300	0.1337	0.1225	0.1232	0.1026	0.0385	......	......	......	......	......	......	......	......	Scale lines on top cover-plate 36 in. and 43 in. from base.
241 500	25 000	0.1176	0.1127	0.1304	0.1338	0.1226	0.1234	......	......	......	......	......	......	......	......	......	......	After 5 min.
241 500	25 000	0.1176	0.1128	0.1304	0.1341	0.1227	0.1235	......	......	......	......	......	......	......	......	......	......	After 10 min.
241 500	25 000	0.1178	0.1128	0.1306	0.1342	0.1228	0.1236	......	......	......	......	......	......	......	......	......	......	After 15 min.
241 500	25 000	0.1179	0.1130	0.1306	0.1344	0.1230	0.1238	0.1041	0.0389	......	......	......	......	......	......	......	......	After 20 min.
241 500	25 000	0.1180	0.1130	0.1309	0.1344	0.1231	0.1239	0.1041	0.0392	......	......	......	......	......	......	......	......	After 25 min.
9 660	1 000	0.0044	0.0028	0.0044	0.0068	0.0052	0.0047	0.0171	0.0234	......	......	......	......	......	......	......	......	July 24th.
9 660	1 000	0.0030	0.0015	0.0029	0.0052	0.0038	0.0033	0.0171	0.0181	......	......	......	......	......	......	......	......	" 25th.
48 300	5 000	0.0224	0.0202	0.0236	0.0265	0.0236	0.0233	0.0281	0.0204									
96 600	10 000	0.0462	0.0435	0.0500	0.0528	0.0483	0.0482	0.0418	0.0258	......	......	......	......	......	......	......	......	Temperature: $27\frac{1}{2}$° cent.
144 900	15 000	0.0701	0.0670	0.0759	0.0794	0.0729	0.0731	0.0591	0.0298									
193 200	20 000	0.0934	0.0897	0.1021	0.1060	0.0974	0.0977	0.0694	0.0344									
241 500	25 000	0.1107	0.1120	0.1292	0.1334	0.1204	0.1223	0.1042	0.0388									
241 500	25 000	0.1107	0.1119	0.1292	0.1334	0.1206	0.1224	0.1043	0.0385	......	......	......	......	......	......	......	......	After 5 min. Scale lines on top cover-plate not any more developed.
251 160	26 000	0.1212	0.1163	0.1348	0.1390	0.1253	0.1273	0.1102	0.0396	0.0066	0.0067	0.0062	......	0.0080	0.0078	0.0066	0.0075	
251 160	26 000	0.1214	0.1164	0.1349	0.1390	0.1255	0.1274	0.1131	0.0401	......	......	......	......	......	......	......	......	After 5 min.
251 160	26 000	0.1213	0.1165	0.1349	0.1390	0.1254	0.1274	......	......	......	......	......	......	......	......	......	......	After 10 min. Scaling a little more prominent.
251 160	26 000	0.1214	0.1164	0.1351	0.1393	0.1255	0.1275	0.1127	0.0401	......	......	......	......	......	......	......	......	After 15 min.
251 160	26 000	0.1214	0.1162	0.1354	0.1395	0.1258	0.1277	0.1138	0.0401	......	......	......	......	......	......	......	......	After 20 min.
251 160	26 000	0.1216	0.1167	0.1354	0.1396	0.1258	0.1278	0.1135	0.0401	......	......	......	......	......	......	......	......	After 25 min.
251 160	26 000	0.1218	0.1167	0.1354	0.1398	0.1260	0.1279	......	......	......	......	......	......	......	......	......	......	After 30 min.
251 160	26 000	0.1218	0.1167	0.1354	0.1398	0.1260	0.1279	......	......	......	......	......	......	......	......	......	......	After 35 min.
260 820	27 000	0.1264	0.1208	0.1431	0.1480	0.1317	0.1340	0.1331	0.0431	0.0068	0.0069	0.0063	......	0.0066	0.0064	0.0092	0.0080	
260 820	27 000	0.1264	0.1208	0.1434	0.1481	0.1320	0.1341	......	......	......	......	......	......	......	......	......	......	After 5 min.
260 820	27 000	0.1265	0.1208	0.1442	0.1485	0.1322	0.1344	0.1371	0.0436	......	......	......	......	......	......	......	......	After 10 min.
260 820	27 000	0.1266	0.1209	0.1439	0.1487	0.1324	0.1345	......	......	......	......	......	......	......	......	......	......	After 15 min.
260 820	27 000	0.1267	0.1210	0.1442	0.1490	0.1327	0.1347	0.1405	0.0438	......	......	......	......	......	......	......	......	After 20 min.
260 820	27 000	0.1266	0.1205	0.1444	0.1493	0.1327	0.1347	......	......	......	......	......	......	......	......	......	......	After 25 min.
260 820	27 000	0.1267	0.1207	0.1446	0.1493	0.1329	0.1348	0.1421	0.0441	......	......	......	......	......	......	......	......	After 30 min.
270 480	28 000	0.1308	0.1247	0.1514	0.1556	0.1382	0.1407	0.1709	0.0477	0.0069	0.0072	0.0063	......	0.0092	0.0090	0.0099	0.0085	
270 480	28 000	0.1311	0.1243	0.1551	0.1608	0.1395	0.1422	0.1869	0.0495	......	......	......	......	......	......	......	......	After 5 min.
270 480	28 000	0.1312	0.1241	0.1562	0.1620	0.1400	0.1427	0.1910	0.0503	......	......	......	......	......	......	......	......	After 10 min.
270 480	28 000	0.1309	0.1239	0.1565	0.1623	0.1400	0.1427	0.1946	0.0506	......	......	......	......	......	......	......	......	After 15 min.
270 480	28 000	0.1311	0.1239	0.1569	0.1628	0.1403	0.1430	0.1970	0.0508	......	......	......	......	......	......	......	......	After 20 min.
270 480	28 000	0.1311	0.1239	0.1572	0.1632	0.1404	0.1432	0.1996	0.0509	......	......	......	......	......	......	......	......	After 25 min.
270 480	28 000	0.1310	0.1239	0.1576	0.1637	0.1404	0.1433	0.2010	0.0509	......	......	......	......	......	......	......	......	After 30 min.
270 480	28 000	0.1311	0.1239	0.1581	0.1638	0.1407	0.1435	0.2022	0.0509	......	......	......	......	......	......	......	......	After 35 min.
270 480	28 000	0.1313	0.1239	0.1582	0.1638	0.1408	0.1436	0.2030	0.0509	......	......	......	......	......	......	......	......	After 40 min.
270 480	28 000	0.1318	0.1239	0.1583	0.1640	0.1410	0.1437	0.2041	0.0509	......	......	......	......	......	......	......	......	After 45 min.
270 480	28 000	0.1313	0.1239	0.1584	0.1640	0.1411	0.1437	0.2060	0.0509	......	......	......	......	......	......	......	......	After 50 min.
270 480	28 000	0.1315	0.1240	0.1585	0.1643	0.1412	0.1439	0.2058	0.0509	......	......	......	......	......	......	......	......	After 55 min.
270 480	28 000	0.1316	0.1240	0.1589	0.1645	0.1413	0.1441	0.2063	0.0509	......	......	......	......	......	......	......	......	After 60 min.
270 480	28 000	0.1316	0.1241	0.1589	0.1648	0.1413	0.1441	0.2070	0.0509	......	......	......	......	......	......	......	......	After 65 min.
270 480	28 000	0.1316	0.1241	0.1594	0.1652	0.1414	0.1443	0.2072	0.0509	......	......	......	......	......	......	......	......	After 70 min.
280 140	29 000	0.1353	0.1266	0.1689	0.1700	0.1482	0.1516	0.2750	0.0583	0.0068	0.0073	0.0060	......	0.0105	0.0101	0.0116	0.0092	Scale lines at about 18 in. spacing on web of chan. on N. side.
280 140	29 000	0.1340	0.1245	0.1764	0.1842	0.1500	0.1538	0.3060	0.0615	......	......	......	......	......	......	......	......	After 5 min.
280 140	29 000	0.1338	0.1238	0.1786	0.1865	0.1507	0.1547	0.3237	0.0626	......	......	......	......	......	......	......	......	After 10 min.
280 140	29 000	0.1333	0.1232	0.1802	0.1883	0.1512	0.1552	0.3350	0.0633	......	......	......	......	......	......	......	......	After 15 min.
280 140	29 000	0.1333	0.1228	0.1819	0.1898	0.1518	0.1559	0.3457	0.0638	......	......	......	......	......	......	......	......	After 20 min.
280 140	29 000	0.1331	0.1225	0.1826	0.1910	0.1521	0.1563	0.3521	0.0642	......	......	......	......	......	......	......	......	After 25 min.
280 140	29 000	0.1332	0.1225	0.1839	0.1922	0.1523	0.1568	0.3595	0.0650	......	......	......	......	......	......	......	......	After 30 min.
280 140	29 000	0.1331	0.1225	0.1844	0.1927	0.1527	0.1571	0.3640	0.0655	......	......	......	......	......	......	......	......	After 35 min.
280 140	29 000	0.1331	0.1223	0.1850	0.1930	0.1528	0.1572	0.3670	0.0657	......	......	......	......	......	......	......	......	After 40 min.
280 140	29 000	0.1331	0.1222	0.1854	0.1935	0.1528	0.1574	0.3705	0.0662	......	......	......	......	......	......	......	......	After 45 min.
280 140	29 000	0.1331	0.1220	0.1854	0.1935	0.1529	0.1574	0.3717	0.0664	......	......	......	......	......	......	......	......	After 50 min.
289 800	30 000	0.1110	0.0925	0.2494	0.2658	0.1709	0.1779	0.9950	0.0586									
289 800	28 720	Ultimate Strength According to Corrected Area.																



TEST No. 54.

* Sectional area used in computing unit loading, 9.66 sq. in. Corrected area, 10.09 sq. in. All loadings in pounds per square inch should be adjusted by multiplying by the factor, 0.9574.

DETAILS OF TEST No. 54, TO ACCOMPANY PROGRESS REPORT OF SPECIAL COMMITTEE ON STEEL COLUMNS AND STRUTS.

TABLE 3.

ABSTRACT OF RECORD OF COLUMN TESTS.

TYPES 4 & 4A – 5 & 5A.

Type	Test no.	Section.	Actual Area	Radius of Gyration	Length	$\frac{l}{r}$	Unit Ultimate Strength	Average Ultimate Strength	Deflection in inches at the Middle of Columns at the Reading just before Failure	
	6	Light Sections.	11.69	2.38	9'–11"	50	35700		0.11 N.	0.015 Down.
	36		"	"	"	"	36500	36900	0.02 S.	0.04 Up.
	75		"	"	"	"	38600		0.0480 S.	0.0021 Up.
	13		11.66	"	16'–10 5/16"	85	34000		0.04	0.06 Up.
4	79		11.73	"	"	"	33900	34000	0.1409 N.	0.0345 Up.
	80	2-8" [s x 11.25-lb = 6.70 □"	11.64	"	"	"	34000		0.0626 S.	0.1304 Down.
		1-8" I x 18.00-lb = 5.33								
	26	Total = 12.03 □"	11.69	"	23'–9 5/8"	120	32000		0.15 S.	0.10
	45		11.61	"	"	"	30700	31900	0.409 N.	0.0216
	49		11.68	"	"	"	33100		0.0712 N	0.0094 Up.
	102	Heavy Sections	16.98	2.32	9'–8"	50	30300		0.0158 N.	0.0169 Down.
	166		16.84	"	"	"	29000	29100	0.0504 S.	0.0150 Down.
	169		16.81	"	"	"	28000		0.0311 S.	0.0019 Up.
	152		16.86	"	16'–5 3/16"	85	26000		0.1044 N.	0.0093 Up.
4A	153		16.89	"	"	"	26400	26600	0.2453 S.	0.0261 Down.
	154	2-8" [s x 18.75-lb = 11.02 □"	17.00	"	"	"	27500		0.0835 N.	0.0078 Down.
		1-8" I x 20.50-lb = 6.03								
	132	Total = 17.05 □"	16.98	"	23'–2 7/16"	120	23500		0.4176 S.	0.0052 Down.
	133		17.07	"	"	"	23900	23900	0.2349 S.	0.0068 Up.
	134		16.96	"	"	"	24200		0.5283 N.	0.0616 Down.
	107	Light Sections.	9.62	1.98	8'–3"	50	38000		0.0280 N.	0.0115 Down
	122		9.65	"	"	"	38000	38000	0.0836 N.	0.0287 Down.
	123		9.65	"	"	"	38000		0.0052 S.	0.0209 Down.
	114		8.61	"	14'–0 5/16"	85	36000		0.1510 S.	0.0078 Down.
5	116		9.66	"	"	"	34000	34300	0.1148 S.	0.0085 Up.
	119	Beth. 8" H Col x 32-lb.	9.59	"	"	"	33000		0.1201 S.	0.0376 Up.
		= 9.17 □"								
	110		8.64	"	19'–9 5/8"	120	33900		0.1148 N.	0.0084 Up
	111		8.68	"	"	"	31000	32000	0.2140 N.	0.0136 Down.
	108		8.61	"	"	"	31000		0.1806 S.	0.0110 Up.
	106	Heavy Sections.	17.86	2.09	8'–8 1/2"	50	34000		0.1201 N.	0.0084 Up.
	120		17.82	"	"	"	36100	35400	0.0606 N.	0.0016 Up.
	121		17.86	"	"	"	36000		0.0284 N.	0.0033 Up.
	115		18.05	"	14'–9 5/8"	85	33900		0.0521 N.	0.0151 Down
5A	117		17.73	"	"	"	32000	32300	0.0929 N.	0.0054 Down.
	118	Beth. 8" H Col x 62-lb	17.77	"	"	"	31000		0.0470 S.	0.0277 Up
		= 18.27 □"								
	109		17.76	"	20'–10 13/16"	120	30000		0.1498 N.	0.0037 Up.
	112		17.97	"	"	"	30000	30000	0.2401 S.	0.0219 Down.
	113		17.73	"	"	"	30000		0.1610 N.	0.0221 Down

TABLE 4.

ABSTRACT OF RECORD OF COLUMN TESTS.

TYPES 6 & 6A—10 & 10A.

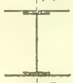
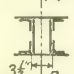
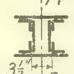
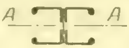
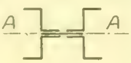
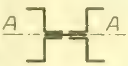
Type	Test no.	Section.	Actual Area	Radius of Gyration	Length	$\frac{l}{r}$	Unit Ultimate Strength.	Average Ultimate Strength.	Deflection in inches at the Middle of Columns at the Reading just before Failure.	
6	10	Light Sections.	13.72	2.31	9′-7½″	50	31200		0.11 N.	0.01 Up.
	40	A  A	13.78	"	"	"	31500	31600	0.0837 S.	0.0065 Up.
	44		13.72	"	"	"	32200		0.0250 N.	0.0020 Up.
	22		13.65	"	16′-4⅜″	85	27800		0.30 N.	0.02 Up.
	63		13.68	"	"	"	29500	29100	0.3040 N.	0.0097 Down.
	64	1-I 10″ x 25-lb. = 7.37 □″	13.60	"	"	"	30100		0.1306 S.	0.0166 Down.
		2 Pls. 11″ x 5/16″ = 6.88								
	31	Total = 14.25 □″	13.73	"	23′-1 3/16″	120	26700		0.50 S.	0.04 Up.
	52		13.75	"	"	"	27700	27200	0.1980 S.	0.0190 Down.
	71		13.65	"	"	"	27300		0.1369 S.	0.0013 Up.
6A	103	Heavy Sections.	23.98	2.47	10′-3½″	50	31200		0.0175 S.	0.0019 Down.
	125	A  A	23.75	"	"	"	32800	32100	0.0078 S.	0.0076 Up.
	128		23.72	"	"	"	32400		0.0496 N.	0.0042 Up.
	155		23.69	"	17′-5 15/16″	85	27200		0.3028 N.	0.0061 Up.
	158		23.73	"	"	"	26800	26800	0.3341 N.	0.0312 Down.
	159	1-I 10″ x 35-lb. = 10.29 □″	23.72	"	"	"	26400		0.3341 S.	0.0522 Down.
		2 Pls. 11″ x 5/8″ = 13.75								
	129	Total = 24.04 □″	23.64	"	24′-8⅜″	120	24900		0.1827 S.	0.0097 Up.
	130		23.83	"	"	"	24000	24800	0.4100 S.	0.0157 Down.
	131		23.76	"	"	"	25500		0.3654 N.	0.0099 Up.
10	3	Light Sections.	10.77	2.33	9′-8½″	50	36100		0.06 S.	0.055 Up.
	4	A  3½″ A	10.80	"	"	"	35700	35800	0.05 N.	0.0251 Up.
	53		10.86	"	"	"	35600		0.0118 N.	0.0217 Up.
	19		10.78	"	16′-6 1/16″	85	32500		0.29 S.	0.200 Up.
	83		10.76	"	"	"	31900	32100	0.0239 N.	0.1117 Down.
	86	2 Pls. 9″ x 1/4″ = 4.500 □″	10.76	"	"	"	31900		0.2611 N.	0.0786 Down.
		4 Ls. 2″ x 2″ x 1/4″ = 3.760								
	23	2 Pls. 5¼″ x 1/4″ = 2.625	10.75	"	23′-3⅝″	120	28400		0.30 N.	0.05 Down
	33	Total = 10.885 □″	10.76	"	"	"	28300	28400	0.23 S.	0.03 Up.
	34		10.75	"	"	"	28400		0.20 S.	0.00
10A	104	Heavy Sections.	18.60	2.34	9′-9″	50	31400		0.0360 S.	0.1556 Down.
	167	A  3½″ A	18.52	"	"	"	32000	31800	0.0099 N.	0.0851 Down.
	170		18.46	"	"	"	32100		0.0783 N.	0.0188 Down.
	144		18.67	"	16′-6⅞″	85	28000		0.0292 S.	0.1044 Down.
	149		18.65	"	"	"	28000	28300	0.1545 S.	0.0992 Down.
	157	2 Pls. 9″ x 7/16″ = 7.88 □″	18.69	"	"	"	29000		0.0731 N.	0.0010 Down.
		4 Ls. 2″ x 2″ x 7/16″ = 6.24								
	140	2 Pls. 5¼″ x 7/16″ = 4.59	18.64	"	23′-4 13/16″	120	27000		0.0392 N.	0.1080 Up.
	141	Total = 18.71 □″	18.80	"	"	"	25900	26300	0.2260 S.	0.1112 Up.
	142		18.65	"	"	"	26000		0.2297 S.	0.1113 Up.

TABLE 5.

ABSTRACT OF RECORD OF COLUMN TESTS.

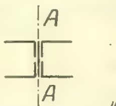
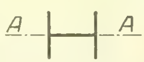
TYPES 7 & 7A – 8 & 8A

Type	Test No.	Section.	Actual Area	Radius of Gyration	Length.	$\frac{l}{r}$	Unit Ultimate Strength.	Average Ultimate Strength.	Deflection in inches at the Middle of Columns at the Reading just before Failure.	
7	5	Light Sections.	13.33	2.48	10'-4"	50	34300		0.00	0.03 Down.
	37		13.30	"	"	"	32700	33400	0.02 S.	0.04 Up.
	76		13.33	"	"	"	33200		0.0034 N.	0.1628 Down.
	15		13.40	"	17'-6 13/16"	85	34000		0.10 S.	0.40 Up.
	89		13.43	"	"	"	30000	31600	0.1081 N.	0.2923 Up.
	90	4-5" Bulb L's x 10.1-lb. = 11.88□"	13.29	"	"	"	30700		0.0914 N.	0.1670 Up.
		1 Pl. 6"x5/16" = 1.88								
	28	Total = 13.76□"	13.36	"	24'-9 5/8"	120	28300		0.05 S.	0.18 Up.
	48		13.34	"	"	"	26900	28100	0.0137 N.	0.3020 Down.
	51		13.33	"	"	"	29000		0.0020 N.	0.0160 Up.
7A*		Heavy Sections.								
										
		4-5" Bulb L's x 13.50-lb. = 15.88□"								
		1 Pl. 6"x7/16" = 2.63								
		Total = 18.51□"								
8	9	Light Sections.	10.98	2.46	10'-3"	50	35900		0.00	0.03 Down.
	39		11.01	"	"	"	35100	35700	0.06 S.	0.06 Up.
	78		11.03	"	"	"	36200		0.0072 S.	0.0086 Up.
	16		11.09	"	17'-5 1/8"	85	31400		0.03 S.	0.11 Up.
	81		11.05	"	"	"	32900	32800	0.0324 N.	0.0653 Up.
	82	4-4" Z's x 1/4" = 9.64□"	11.11	"	"	"	34000		0.0117 S.	0.1597 Up.
		1 Pl. 7"x1/4" = 1.75								
	27	Total = 11.39□"	11.06	"	24'-7 3/16"	120	29100		0.07 N.	0.06 Down.
	46		11.11	"	"	"	29400	29700	0.0417	0.0194 Up.
	50		11.16	"	"	"	30400		0.0088 N.	0.1281 Down.
8A	100	Heavy Sections	25.78	2.51	10'-5 1/2"	50	33000		0.0235 N.	0.0431 Down.
	126		25.86	"	"	"	33500	32900	0.0313 N.	0.0940 Down.
	127		25.83	"	"	"	32200		0.0261 N.	0.2400 Down.
	156		25.56	"	17'-9 3/8"	85	31300		0.0835 S.	0.2297 Up.
	160		25.57	"	"	"	25300	31400†	0.0010 S.	0.3393 Up.
	161	4-4" Z's x 5/8" = 22.20□"	25.43	"	"	"	31500		0.0459 N.	0.1462 Up.
		1 Pl. 7"x5/8" = 4.38								
	162	Total = 26.58□"	25.66	"	25'-1 3/16"	120	27500		0.1019 S.	0.3075 Up.
	163		25.70	"	"	"	27000	27300	0.0848 N.	0.2025 Down.
	164		25.68	"	"	"	27500		0.0032 N.	0.760 Down.

* Tests have not been made. † Test no. 160 omitted in determining the average.

TABLE 6.

ABSTRACT OF RECORD OF COLUMN TESTS.
TYPES I & IA AND CARNEGIE SECTIONS.

Type	Test no.	Section.	Actual Area	Radius of Gyration	Length	$\frac{l}{r}$	Unit Ultimate Strength	Average Ultimate Strength	Deflection in inches at the Middle of Columns at the Reading just before Failure.	
I	178	Light Sections.	11.93	2.24	3'–8 1/16"	20	38400		0.0261 S.	0.0261 Up.
	179		11.81	"	"	"	38200	38200	0.0040 N.	0.0580 Down.
	180		11.79	"	3'–8 3/4"	"	38000		0.0068 S.	0.0042 Down.
		4 L's 5"×3"×5/16" = 9.60 □"								
		1 Pl. 6"×5/16" = 1.875 □"								
		Total = 11.475 □"								
IA	175	Heavy Sections.	22.26	2.36	3'–11 1/16"	20	43200		0.0282 N.	0.0032 Down.
	176		22.33	"	"	"	43300	43600	0.0578 N.	0.1023 Down.
	177		22.33	"	"	"	44300		0.0385 N.	0.0462 Up.
		4 L's 5"×3"×5/8" = 18.44 □"								
		1 Pl. 6"×5/8" = 3.75 □"								
		Total = 22.19 □"								
		Light Sections.	7.00	1.45	6'–0 7/16"	50	31500			
			7.00	"	6'–0 1/2"	"	30000	31000		
			7.00	"	"	"	31400			
			7.00	1.45	10'–3 1/4"	85	30600			
			7.00	"	10'–3 3/16"	"	30000	30400		
		Carnegie 6" H Col. × 23.8-lb. = 7.00 □"	7.00	"	10'–3 1/4"	"	30500			
			7.00	1.45	14'–6"	120	26400			
			7.00	"	14'–6 3/16"	"	29300	27200		
			7.00	"	14'–6"	"	26000			
		Light Sections.	10.00	1.87	7'–9 1/2"	50	32800			
			10.00	"	7'–9 3/8"	"	33900	33500		
			10.00	"	"	"	33800			
			10.00	1.87	13'–3"	85	31800			
			10.00	"	"	"	31000	31700		
		Carnegie 8" H Col. × 34.0-lb. = 10.00 □"	10.00	"	"	"	32300			
			10.00	1.87	18'–8 1/2"	120	30000			
			10.00	"	"	"	28900	29200		
			10.00	"	"	"	28700			

Areas given for Carnegie Sections are nominal.

TABLE 7.

ABSTRACT OF RECORD OF COLUMN TESTS.

TYPES 5, 5A & 5B.

Type.	Test no.	Section.	Actual Area.	Radius of Gyration	Length.	$\frac{l}{r}$	Unit Ultimate Strength.	Average Ultimate Strength.	Deflection in inches at the Middle of Columns at the Reading just before Failure.	
		LIGHT AND HEAVY SECTIONS.								
	107	Light Sections.	9.62	1.98	8′-3″	50	38000		0.0280 N.	0.0115 Down.
	122		9.65	"	"	"	38000	38000	0.0836 N.	0.0287 Down.
	123		9.65	"	"	"	38000		0.0052 S.	0.0209 Down.
	114		8.61	"	14′-0 5/16″	85	36000		0.1510 S.	0.0078 Down.
5	116		9.66	"	"	"	34000	34300	0.1148 S.	0.0085 Up.
	119	Beth. 8″ H Col. x 32-lb.	9.59	"	"	"	33000		0.1201 S.	0.0376 Up.
		= 9.17□″								
	110		8.64	"	19′-9 5/8″	120	33900		0.1148 N.	0.0084 Up.
	111		8.68	"	"	"	31000	32000	0.2140 N.	0.0136 Down.
	108		8.61	"	"	"	31000		0.1806 S.	0.0110 Up.
	106	Heavy Sections.	17.86	2.09	8′-8½″	50	34000		0.1201 N.	0.0084 Up.
	120		17.82	"	"	"	36100	35400	0.0606 N.	0.0016 Up.
	121		17.86	"	"	"	36000		0.0284 N.	0.0033 Up.
	115		18.05	"	14′-9 5/8″	85	33900		0.0521 N.	0.0151 Down.
5A	117		17.73	"	"	"	32000	32300	0.0929 N.	0.0054 Down.
	118	Beth. 8″ H Col. x 62-lb.	17.77	"	"	"	31000		0.0470 S.	0.0277 Up.
		= 18.27□″								
	109		17.76	"	20′-10 13/16″	120	30000		0.1498 N.	0.0037 Up
	112		17.97	"	"	"	30000	30000	0.2401 S.	0.0219 Down.
	113		17.73	"	"	"	30000		0.1610 N.	0.0221 Down.
		EXTRA HEAVY SECTIONS.								
	225	Extra Heavy Sections	27.49	2.17	9′-0½″	50	24000		0.0825 S.	0.0104 Down.
	226		27.18	"	"	"	24600	25200	0.1044 S.	0.0261 Down.
	182		27.50	"	"	"	27100		0.0522 N.	0.0033 Up.
	227		27.28	"	15′-4 7/16″	85	22600		0.2297 N.	0.0438 Down.
5B	228		27.27	"	"	"	23000	23500	0.0731 S.	0.0078 Up.
	183	Beth. 8″ H Col. x 91-lb.	27.45	"	"	"	24800		0.0553 S.	0.0016 Down.
		= 26.64□″								
	223		27.50	"	21′-8 3/8″	120	21000		0.3341 S.	0.0113 Up.
	224		27.54	"	"	"	21000	21300	0.2610 S.	0.0459 Up.
	181		27.57	"	"	"	21800		0.1903 S.	0.0031 Up.

TABLE 8.

ABSTRACT OF RECORD OF COLUMN TESTS.

TYPES I, IA & IB.

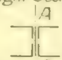
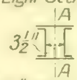
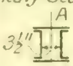
Type.	Test no.	Section.	Actual Area	Radius of Gyration	Length.	$\frac{l}{r}$	Unit Ultimate Strength	Average Ultimate Strength	Deflection in inches at the Middle of Columns at the Reading just before Failure.	
		LIGHT AND HEAVY SECTIONS.								
I	8	Light Sections.	11.13	2.24	9'-4"	50	32400		0.0350 S.	0.0000
	38	A	11.17	"	"	"	32800	32700	0.0700 S.	0.1000 Up.
	77		11.13	"	"	"	33000		0.0500 N.	0.0313 Up.
	18		11.49	"	15'-10 7/16"	85	32100		0.1200 S.	0.0500 Up.
	84	A	11.49	"	"	"	31400	31200	0.0252 S.	0.1281 Down.
	85	4 Ls 5"x3"x5/16" = 9.60 □"	11.35	"	"	"	30000		0.0626 N.	0.1368 Up.
		1 Pl. 6"x5/16" = 1.875								
	30	Total = 11.475 □"	11.20	"	22'-4 13/16"	120	28600		0.0000	0.0600 Down.
	56		11.14	"	"	"	27600	28300	0.1175 S.	0.1091 Up.
	69		11.30	"	"	"	28700		0.0135 N.	0.1785 Down.
IA	93	Heavy Sections.	22.25	2.36	9'-10"	50	29000		0.0080 S.	0.0574 Down.
	96		22.13	"	"	"	28900	29200	0.0731 N.	0.0457 Up.
	97	A  A	22.28	"	"	"	29700		0.0653 N.	0.0002 Up.
	91		22.134	"	16'-8 5/8"	85	28000		0.0475 S.	0.0883 Up.
	94		21.92	"	"	"	28200	28100	0.4053 N.	0.2636 Up.
	95	4 Ls 5"x3"x5/8" = 18.44 □"	22.05	"	"	"	28000		0.0764 N.	0.0825 Down.
		1 Pl. 6"x5/8" = 3.75								
	92	Total = 22.19 □"	22.15	"	23'-7 3/16"	120	25000		0.1430 S.	0.2923 Down.
	98		22.10	"	"	"	25500	25400	0.3863 N.	0.0762 Up.
	99		22.10	"	"	"	25800		0.1326 N.	0.0606 Down.
		EXTRA HEAVY SECTIONS.								
IB		Extra Heavy Sections.								
		A  A								
	229		28.43	2.29	16'-2 5/8"	85	28200		0.0668 N.	0.0167 Down.
	230		28.37	"	"	"	27000	27700	0.0900 S.	0.0475 Down.
	231	4 Ls 5"x3"x13/16" = 23.36 □"	28.53	"	"	"	28000		0.0691 N.	0.0007 Up.
		1 Pl. 6x7/8" = 5.25								
		Total = 28.61 □"								

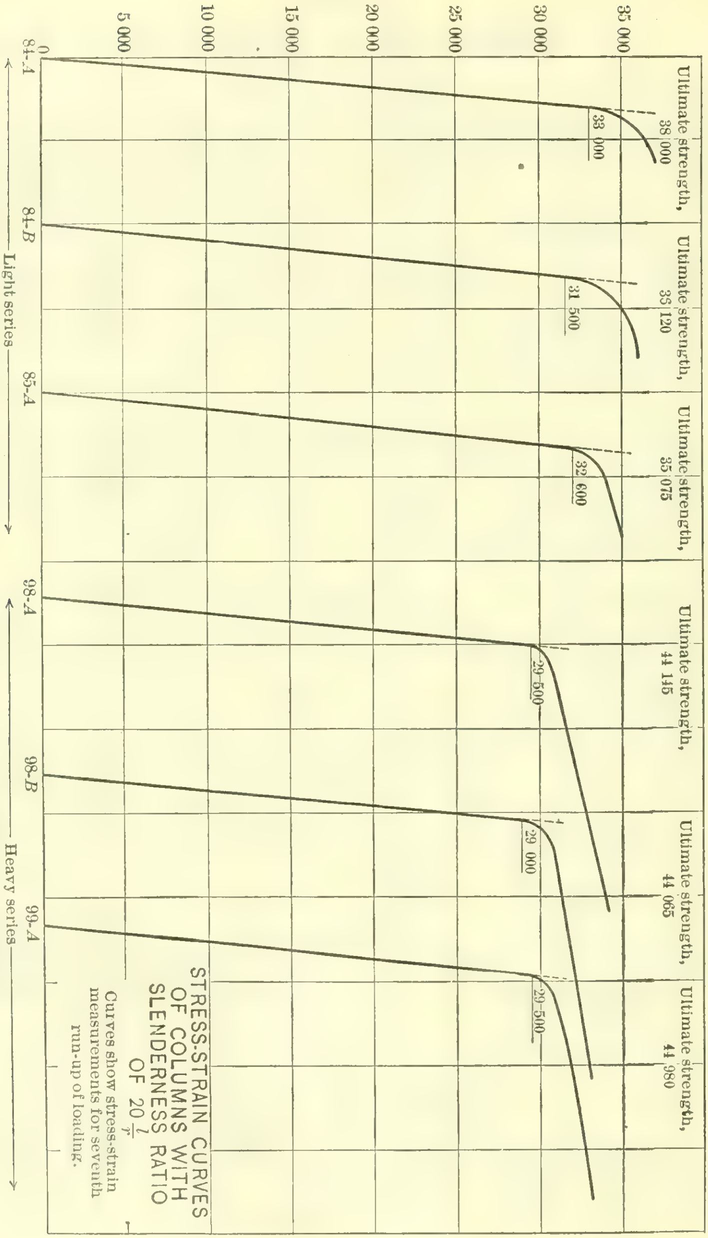
TABLE 9.

ABSTRACT OF RECORD OF COLUMN TESTS.

TYPES 1 & 1A–2 & 2A–4 & 4A.

Slenderness Ratio $\frac{l}{r}$ = 155.

Type	Test no.	Section.	Actual Area.	Radius of Gyration	Length.	$\frac{l}{r}$	Unit Ultimate Strength.	Average Ultimate Strength.	Deflection in inches at the Middle of Columns at the Reading just before Failure.	
		Light Sections. 								
	207		11.84	2.24	28′–11$\frac{3}{16}$″	155	27000		0.4176 S.	0.2506 Up.
1	218		11.69	"	"	"	25600	26200	0.1629 S.	0.6277 Down.
	222	4 Ls 5″×3″×$\frac{5}{16}$″ = 9.60 □″	11.68	"	"	"	26000		0.1201 N.	0.3289 Down.
		1 Pl. 6″×$\frac{5}{16}$″ = 1.875								
		Total = 11.475 □″								
		Heavy Sections. 								
	205		22.35	2.36	30′–5$\frac{13}{16}$″	155	22200		0.1629 S.	0.3967 Up
1A	206		22.35	"	"	"	23200	22700	0.0992 S.	0.4594 Down.
	208	4 Ls 5″×3″×$\frac{5}{8}$″ = 18.44 □″	22.31	"	"	"	22700		0.2514 S.	0.4802 Down.
		1 Pl. 6″×$\frac{5}{8}$″ = 3.75								
		Total = 22.19 □″								
		Light Sections. 								
	215		10.39	2.31	29′–10$\frac{1}{16}$″	155	26100		0.8352 N.	0.1284 Up.
2	217		10.42	"	"	"	27800	26500	0.5116 N.	0.0668 Down.
	221	2–6″ [s × 10.5-lb = 6.18 □″	10.46	"	"	"	25600		0.9271 S.	0.0418 Down.
		2 Pls 8″×$\frac{1}{4}$″ = 4.00								
		Total = 10.18 □″								
		Heavy Sections. 								
	214		16.75	2.33	30′–1$\frac{1}{8}$″	155	24700		0.4959 S.	0.2589 Down.
2A	216		16.87	"	"	"	25000	24900	0.1958 N.	0.6160 Up.
	219	2–6″ [s × 15.5-lb = 9.12 □″	16.81	"	"	"	25000		0.0428 S.	0.2714 Down.
		2 Pls 8″×$\frac{1}{2}$″ = 8.00								
		Total = 17.12 □″								
		Light Sections. 								
	211		11.81	2.38	30′–8$\frac{7}{8}$″	155	24000		0.5324 N.	0.0002 Down.
4	212		11.93	"	"	"	23200	23600	0.7465 N.	0.0491 Down.
	220	2–8″ [s × 11.25-lb = 6.70 □″	11.84	"	"	"	23500		0.5116 S.	0.0040 Down.
		1–8″ I × 18.00-lb = 5.33								
		Total = 12.03 □″								
		Heavy Sections. 								
	209		16.74	2.32	29′–11$\frac{5}{8}$″	155	24000		0.1754 S.	0.0569 Up.
4A	210		16.67	"	"	"	23000	23300	0.2453 S.	0.0512 Up.
	213	2–8″ [s × 18.75-lb = 11.02 □″	16.33	"	"	"	23000		0.1973 S.	0.0063 Up.
		1–8″ I × 20.50-lb = 6.03								
		Total = 17.05 □″								



Nominal area, 11.475 sq. in.

Nominal area, 22.190 sq. in.

FIG. 5.

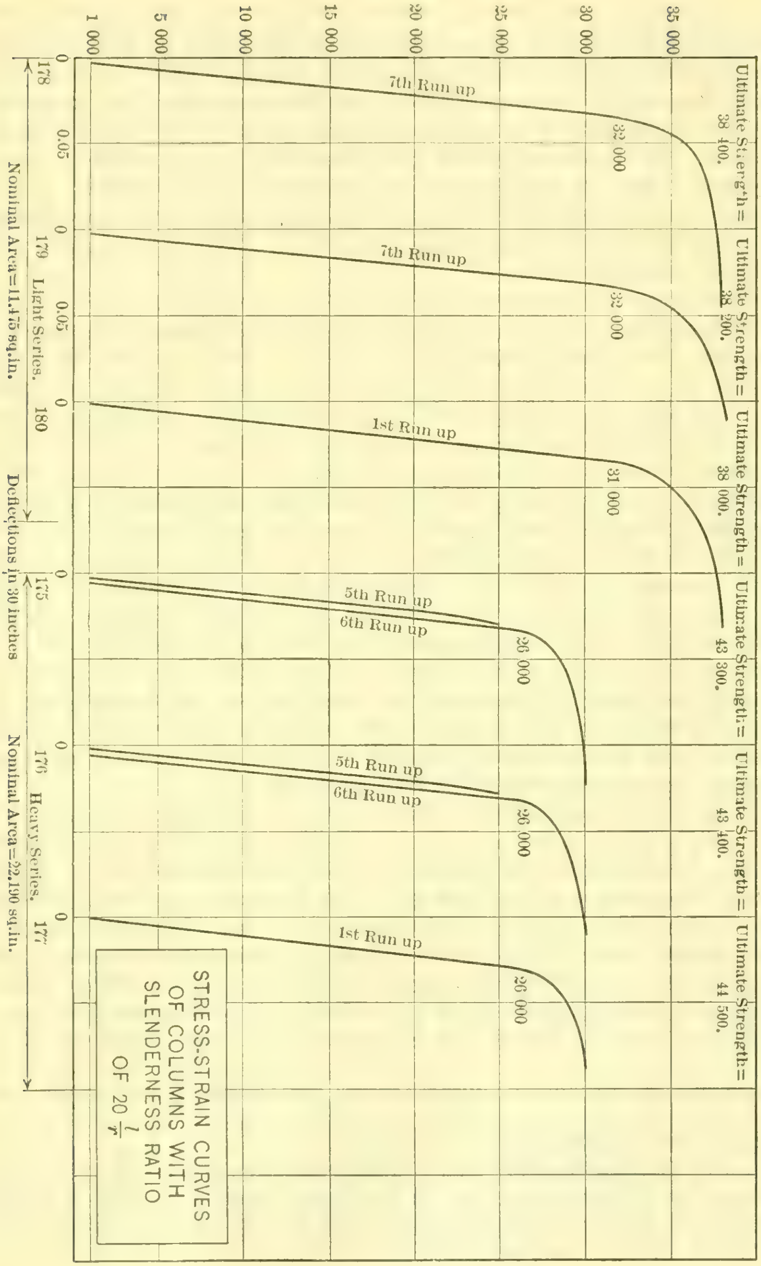

STRESS-STRAIN CURVES OF COLUMNS WITH SLENDERNESS RATIO OF 20 $\frac{l}{r}$
Ultimate Strength = 38 400.
Ultimate Strength = 38 200.
Ultimate Strength = 38 000.
Ultimate Strength = 43 300.
Ultimate Strength = 43 400.
Ultimate Strength = 41 500.
7th Run up
7th Run up
1st Run up
5th Run up
6th Run up
5th Run up
6th Run up
1st Run up
32 000
32 000
31 000
26 000
26 000
26 000
1 000
5 000
10 000
15 000
20 000
25 000
30 000
35 000
0
0.05
178
179
180
175
176
177
Light Series.
Heavy Series.
Nominal Area = 11.475 sq.in.
Deflections in 30 inches
Nominal Area = 22.190 sq.in.

FIG. 6.

A question arose as to the tendency of the material to flow under load, and in order to secure information on this point, a special time test was inaugurated. This is given as No. 54, where it will be seen that various loadings were applied and then held while the compressometers were read at intervals of 5 min. until the flow of the metal ceased. Although the results are interesting, and might be a matter of speculation in beginning a new programme, the variation in the failure point from similar columns does not seem to show enough difference to warrant any extensive carrying out of time tests in the present programme.

As stated in the foregoing report, the programme was arranged "with the intention, as far as practicable, of eliminating all variables except that of form, and was subdivided so as to consider the effect of light and heavy sections." The Committee felt at that time that it was better to confine itself to a rather narrow programme, with the use of columns having a cross-section arranged so as to avoid lacing or battens, rather than to attempt to cover the effects of details and unsymmetrical sections. The Committee was influenced somewhat in the choice of certain forms of columns by an opinion held by some engineers that columns failed largely through the crippling of the thin outstanding legs. For example, in a discussion published in 1908,* George E. Gifford, M. Am. Soc. C. E., expressed the opinion that angles $\frac{1}{2}$ in. thick would be more than twice as strong as angles $\frac{1}{4}$ in. thick; and the late T. H. Johnson, M. Am. Soc. C. E., suggested the use of bulb-angles, believing that the bulb on the outstanding leg gave stiffness to the columns. He called attention to the superior behavior of Z-bar columns, "evidently due to the support of the horizontal leg afforded by the outer leg of the Z".

On this account, the Committee selected a bulb-angle column, Type 7, and a Z-bar column, Type 8, for test, and designed a column, Type 6, made up of an I-beam with two plates riveted to the flanges. The Committee realized that this Type 6 column was not a satisfactory form, but wanted to study the effect of the outstanding edges of the flange plates. Tables 1 to 9 show the record of the column tests published in the Progress Reports of 1916 and 1917. These records call attention to the decrease in unit ultimate strength of the heavy sections when compared with the light ones. This, with the single exception of Type 6, slenderness ratio $\frac{l}{r} = 50$ (see Table 4) was true for all the different types of columns.

The results, as shown by the foregoing tests, though demonstrating the value of a thicker plate in the short columns of Type 6, hardly prove or disprove the original question before the Committee, as to

* *Proceedings*, Am. Soc. for Testing Materials.

the influence of the thin outstanding legs. It is possible that, for very thin metal, the ratio of the thickness of the outstanding leg to its width might have an effect on the strength of the column, but, for the thicknesses of the material which have been used in the programme of tests outlined by the Committee, it is evident that the ratio of thickness to width is not a controlling factor.

In seeking to account for the falling off in strength of the heavy material, the Committee learned that it was necessary to look beyond the differences in the ratios of widths to thicknesses of outstanding legs, and variations of cross-section. The only remaining element which could be charged with responsibility appeared to be the metal itself, and though the intention had been to secure material of a uniform grade, a more careful and thorough investigation disclosed the fact that the attempt had not been successful.

The physical characteristics and strength of structural steel are affected by the amount of working which it receives in passing through the rolls, and, for the same size of ingot, the heavy material does not receive as much working as the light; consequently, for the same chemical composition, the heavier material is weaker. In ordering the steel for the column tests, the Committee provided for this difference in working by allowing the mill to change the carbon content so as to endeavor to produce the same strength for the light material as for the heavy. When the attention of the Committee and the Bureau of Standards was drawn to the falling off in the unit strength of the heavy material, the Bureau of Standards investigated the record of the physical and chemical tests made at the mill when the steel was rolled. The complete record of these physical and chemical tests is shown in Table 10, and it will be seen that this record gives no indication of the difference in strength between the heavy and light material.

A note concerning the choice of the original specifications might be in order before commenting on the record of specimen tests.

At the time the Bureau of Standards offered to make the column tests, Dr. Stratton requested the Committee to write the specifications and secure prices. It was desired to have the material for the columns as nearly uniform as possible, the thought being to have as much of it as possible come from a single heat. The specifications (page 2417) are the result of the conference of the Committee with the steel manufacturers, and, as stated before, they are very much more rigid than the average structural specifications. The Committee accepted some material which did not fully meet the rigid specifications, but it was considered at that time to be fully equal to, or better than, the ordinary grade of structural steel, which has an allowable variation of 5 000 lb. per sq. in., more or less, from a desired ultimate strength of 60 000 lb. per sq. in.

TABLE 10.—(*Continued.*)

Heat No.	Type.	Lot.	Specimen cut from :	Elastic limit, in pounds per square inch.	Ultimate strength, in pounds per square inch.	Percentage of elongation in 8 in.	Percentage of reduction of area.	Percentage of: Carbon.	Manganese.	Phosphorus.	Sulphur.	Character of fracture.
Specifications.				37 000 to 39 000	58 500 to 61 500	26 to 30	52 to 60	variable.	0.40 to 0.50	0.00 to 0.03	0.00 to 0.04	Silky. Cup.
14414	2a	3	8 x ½	40 160*	61 360	32.5*	60.8*	0.21	0.44	0.010	0.038	½ Cup.
C11393	2a	3	6 Ch. 15½-lb.	35 430*	58 180*	30.0	52.6	0.24	0.44	0.010	0.041*	Angular.
47096	3	1	9½ x ¼	37 670	54 980*	29.2	57.4	0.17	0.44	0.014	0.029	½ Cup.
47096	3	1	9½ x ¼	36 550*	53 500*	30.6*	46.8*	0.17	0.44	0.014	0.029	½ Cup.
46201	3	1	5-in. Ch. 6½	42 020*	59 240	23.2*	67.2*	0.20	0.48	0.020	0.035	½ Cup.
46201	3	1	do	44 700*	55 900*	26.3	57.7	0.20	0.48	0.020	0.035	Cup.
47096	3	1	8 x ¼	38 300	54 250*	25.0*	48.8*	0.17	0.44	0.014	0.029	Angular.
38054	3a	2	5-in. x 11.5-lb. Ch.	36 790*	58 100*	34.7*	59.6	0.20	0.40	0.020	0.032	Angular.
41029	3a	2	9½ x ½	39 000	60 350	31.2*	54.7	0.22	0.44	0.027	0.036	Angular.
44200	4	1	8-in. Ch. 11¼	40 800*	58 600	27.8	48.7*	0.20	0.45	0.018	0.028	Irregular.
44200	4	1	do	38 650	60 700	25.0*	56.2	0.20	0.45	0.018	0.028	½ Cup.
27225	4	1	8-in. I 18-lb.	34 960*	58 280*	28.5	58.2	....	....	.....	.....	Angular.
27225	4	1	do	32 100*	54 600*	30.0	56.4	....	....	.....	.....	½ Cup.
C4289	4	3	8-in. Ch. 11¼-lb.	36 260*	58 240*	26.2	57.9	0.21	0.47	0.012	0.039	Irregular.
10184	2a	2	8 x ½ Pl.	37 430	60 460	27.5	51.1*	0.22	0.50	0.011	0.040	Angular.
10184	2a	2	do	37 420	61 620*	27.5	52.2	0.22	0.50	0.011	0.040	Angular.
48332	4a	2	8-in. x 18¾-lb. Ch.	38 590	59 240	30.5*	49.6*	....	....	.....	.....	½ Cup.
48332	4a	2	do	38 020	58 880	30.5*	49.8*	....	....	.....	.....	Cup.
C5315	4a	3	8-in. I 20.5-lb.	36 990*	59 700	30.0	57.9	0.23	0.47	0.011	0.037	Angular.
42239	4a	2 or 3	8-in. 20½-lb. I	37 990	59 740	28.7	54.0	0.21	0.50	0.010	0.036	do.
42239	4a	2 or 3	do	36 370*	59 600	27.7	53.6	0.21	0.50	0.010	0.036	do.
47096	6	1	11 x 5/16	35 140*	54 740*	30.7*	55.9	0.17	0.44	0.014	0.029	Angular.
47096	6	1	11 x 5/16	32 550*	53 200*	29.4	54.9	0.17	0.44	0.014	0.029	Angular.
42171	6	1	11 x 5/16	37 570	58 100*	30.5*	60.1*	0.20	0.44	0.010	0.028	½ Cup.
42171	6	1	11 x 5/16	32 990	57 830*	31.4*	54.5	0.20	0.44	0.010	0.028	Angular.
39235	6	1	10 I 25-lb.	38 580	63 600*	28.7	55.0	....	....	.....	.....	Angular.
39235	6	1	do	35 370*	62 590*	26.2	58.2	....	....	.....	.....	Angular.
39235	6	1	do	32 300*	59 800*	27.0	52.8	....	....	.....	.....	½ Cup.
37261	6	1	do	37 680	55 400*	30.0	61.6*	....	....	.....	.....	Angular.
37261	6	1	do	29 100*	53 200*	32.0*	54.4	....	....	.....	.....	Cup.
41029	6a	2	11 x ⅝	39 090*	60 540	28.0	53.9	0.22	0.44	0.027	0.036	Angular.
42239	6a	2	10-in. — 35-lb.	37 930	60 300	28.0	47.2*	0.21	0.50	0.010	0.036	Cup.
42239	6a	2	do	37 930	60 250	26.2	46.8*	0.21	0.50	0.010	0.036	Irregular.
												Angular.

* Not within limits of specifications.

TABLE 10.—RECORD OF PHYSICAL AND CHEMICAL SPECIMEN TESTS MADE BY MILL AT PITTSBURGH.

Heat No.	Type.	Lot.	Specimen cut from :	Elastic limit, in pounds per square inch.	Ultimate strength, in pounds per square inch.	Percentage of elongation in 8 in.	Percentage of reduction of area.	Percentage of: Carbon.	Manganese.	Phosphorus.	Sulphur.	Character of fracture.
Specifications.				37 000 to 39 000	58 500 to 61 500	26 to 30	52 to 60	variable.	0.40 to 0.50	0.00 to 0.03	0.00 to 0.04	Silky. Cup.
11105	1 & 7	1	6 x 5/16	36 950*	60 860	27.5	57.0	0.20	0.47	0.013	0.038	Angular.
11105	1 & 7	1	6 x 5/16	35 480*	59 680	25.5*	57.3	0.20	0.47	0.013	0.038	do.
11105	1 & 7	1	6 x 5/16	34 450*	60 260	28.5	50.5*	0.20	0.47	0.013	0.038	do.
42171	1	1	5 x 3 x 5/16 ∠	34 270*	57 990*	28.7	55.1	0.20	0.44	0.010	0.028	½ Cup.
42171	1	1	5 x 3 x 5/16 ∠	34 070*	57 740*	28.0	55.0	0.20	0.44	0.010	0.028	Angular.
42171	1	1	5 x 3 x 5/16	29 700*	56 000*	28.0	51.4*	0.20	0.44	0.010	0.028	Angular.
11105	1 & 7	1	6 x 5/16	32 950*	59 730	29.5	54.8	0.20	0.47	0.013	0.038	do.
C13333	1	3	5 x 3 x 5/16 ∠	36 230*	61 600*	27.5	51.8*	0.22	0.47	0.037*	0.047*	Angular.
C13333	1	3	5 x 3 x 5/16	36 240*	58 050*	27.5	50.7*	0.22	0.47	0.037*	0.047*	Cup.
C14304	1	3	6 x 5/16	37 040	59 360	27.5	56.0	0.25	0.50	0.012	0.040	Angular.
C1267	1	3	5 x 3 x 5/16	36 350*	60 860	30.0	60.9*	0.21	0.42	0.016	0.040	Cup.
C13354	1*a*	2 or 3	5 x 3 x 5/8	38 800	60 920	27.5	58.2	0.21	0.50	0.010	0.032	½ Cup.
C13354	1*a*	2 or 3	5 x 3 x 5/8	38 280	61 430	30.0	58.4	0.21	0.50	0.010	0.032	½ Cup.
38054	1*a*	2	6 x 5/8	36 940*	58 400*	35.5*	55.6	0.20	0.40	0.020	0.032	Angular.
1257	1*a*	2	5 x 3 x 5/8	38 300	60 460	28.7	44.2*	0.22	0.50	0.015	0.030	Angular.
1257	1*a*	2	5 x 3 x 5/8	39 100*	58 970	27.5	49.3*	0.22	0.50	0.015	0.030	Angular.
10032	1*a*	2	5 x 3 x 5/8	38 280	57 800*	30.0	53.3	0.20	0.42	0.012	0.039	Cup.
10032	1*a*	2	5 x 3 x 5/8	38 240	58 760	30.0	59.3	0.20	0.42	0.012	0.039	Cup.
C1267	1*a*	3	5 x 3 x 5/8	36 600*	59 380	28.7	53.4	0.21	0.42	0.016	0.040	Angular.
C10133	1*a*	3	6 x 5/8	37 190	61 460	27.5	55.6	0.20	0.46	0.040*	0.037	Angular.
C5400	1*b*	3	6 x 7/8	36 880*	59 520	27.5	47.3*	0.21	0.47	0.022	0.032	Cup.
C1267	1*b*	3	5 x 3 x 13/16	35 910*	59 580	28.7	56.3	0.21	0.42	0.016	0.040	Angular.
C1267	1*b*	3	5 x 3 x 13/16	36 260*	59 740	28.7	58.9	0.21	0.42	0.016	0.040	Angular.
C1267	1*b*	3	5 x 3 x 13/16	36 650	59 480	30.0	59.7	0.21	0.42	0.016	0.040	Cup.
31082	2	1	6 x 10½-lb. Ch.	35 270*	57 630*	30.2*	61.3*	....	....	.....	.....	½ Cup.
31082	2	1	do	36 500	59 700	28.6	57.7	....	....	.....	.....	Cup.
47096	2	1	8 x ¼	38 670	55 520*	31.2*	54.5	0.17	0.44	0.014	0.029	½ Cup.
42171	2	1	8 x ¼	37 840	58 130*	30.0	58.5	0.20	0.44	0.010	0.028	Angular.
42171	2	1	8 x ¼	34 310*	57 550*	30.0	49.8*	0.20	0.44	0.010	0.028	Angular.
14114	2	3	8 x ¼	40 510*	63 660*	27.5	61.1*	0.21	0.44	0.010	0.038	Cup.
C11393	2	3	6 Ch. 10½-lb.	35 900*	61 700*	27.5	54.5	0.24	0.44	0.010	0.040	Angular.
38054	2*a*	2	6 15½-lb. Ch.	36 500*	56 990*	30.2*	51.7*	0.20	0.40	0.020	0.032	Angular.
38054	2*a*	2	6 15½-lb. Ch.	36 220*	59 840	27.5	47.3*	0.20	0.40	0.020	0.032	Cup.

* Not within limits of specifications.

TABLE 10 (*Continued*)—REJECTED MATERIAL.

Heat No.	Type.	Lot.	Specimen cut from :	Elastic limit, in pounds per square inch.	Ultimate strength, in pounds per square inch.	Percentage of elongation in 8 in.	Percentage of reduction of area.	Percentage of : Carbon.	Manganese.	Phosphorus.	Sulphur.	Character of fracture.
	Specifications			37 000 to 39 000	58 500 to 61 500	26 to 30	52 to 60	variable	0.40 to 0.50	0.00 to 0.03	0.00 to 0.04	Silky. Cup.
31215	....	....	5 x 3 x 5/8	36 540*	54 860*	32.5*	60.0	0.21	0.48	0.016	0.032	½ Cup.
31215	....	....	do.	36 470*	54 570*	31.2*	58.7	0.21	0.48	0.016	0.032	½ Cup.
31215	....	....	do.	36 010*	54 060*	32.5*	60.1*	0.21	0.48	0.016	0.032	½ Cup.
34428	....	....	8 lb. x 18¾ lb. Ch.	37 550	55 340*	29.7	52.7	0.20	0.49	0.019	0.033	Angular.
34228	....	....	do.	36 650*	55 320*	30.0	52.3	0.20	0.49	0.019	0.033	do.
44292	....	....	do.	36 290*	56 420*	31.0*	52.7	0.20	0.40	0.010	0.031	Irregular.
44292	....	....	do.	38 560	55 400*	30.7*	48.8*	0.20	0.40	0.010	0.031	Angular.
39208	....	....	8 in. I 18		64 320*	26.0	49.5*	....	....	.....	.....	Irregular.
39208	....	....	8 in. I 15		63 580*	26.2	51.8*	....	....	.....	.....	½ Cup.
39208	....	....	8 in. I 18	34 100*	61 300	27.0	51.5*	....	....	.....	.....	Angular.
47250	....	....	10 in. I 25	36 360*	56 450*	32.2*	59.5	....	....	.....	.....	Cup.
47250	....	....	10 in. I 25	31 200*	54 600*	30.0	58.2	....	....	.....	.....	Cup.

* Not within limits of specification.

TABLE 10.—(*Continued.*)

Heat No.	Type.	Lot.	Specimen cut from :	Elastic limit, in pounds per square inch.	Ultimate strength, in pounds per square inch.	Percentage of elongation in 8 in.	Percentage of reduction of area.	Percentage of: Carbon.	Manganese.	Phosphorus.	Sulphur.	Character of fracture.
Specifications.				37 000 to 39 000	58 500 to 61 500	20 to 30	52 to 60	variable.	0.40 to 0.50	0.00 to 0.03	0.00 to 0.04	Silky. Cup.
42171	7?	1	5-in. bulb 10.1-lb.	33 100*	56 300*	28.0	50.8*	0.20	0.44	0.010	0.028	Angular.
47096	8	1	7 x ¼ Pl.	39 860*	55 490*	31.5*	50.8*	0.17	0.44	0.014	0.029	Angular.
47096	8	1	7 x ¼	37 800	53 500*	30.6*	54.3	0.17	0.44	0.014	0.029	Cup.
42171	8	1	4-in. Z 8½-lb.	39 650*	57 640*	31.2*	56.1	0.20	0.44	0.010	0.028	Angular.
42171	8	1	do	39 430*	58 340*	31.2*	59.5	0.20	0.44	0.010	0.028	Irregular.
42171	8	1	do	39 050*	56 700*	32.5*	55.3	0.20	0.44	0.010	0.028	Angular.
42171	8	1	do	39 370*	56 100*	33.0*	55.4	0.20	0.44	0.010	0.028	Irregular.
7463	8a	2	7 x ⅝ Pl.	35 750*	58 240*	27.5	51.1*	0.20	0.46	0.019	0.034	Angular.
7463	8a	2	do	38 740	59 620	28.7	56.0	0.20	0.46	0.019	0.034	Angular.
7463	8a	2	4-in. Z	38 260	58 240*	28.7	52.2	0.20	0.46	0.019	0.036	Angular.
11105	10	1	5¼ x ¼	39 220*	60 060	27.5	55.9	0.20	0.47	0.013	0.038	Angular.
11105	10	1	do	37 400	59 420	29.0	60.7*	0.20	0.47	0.013	0.038	½ Cup.
11105	10	1	do	33 950*	59 510	31.5*	55.8	0.20	0.47	0.013	0.038	Cup.
11105	10	1	do	35 000*	59 960	29.3	52.8	0.20	0.47	0.013	0.038	Angular.
47096	10	1	2 x 2 x ¼∠	36 620*	56 800*	31.2*	61.7*	0.17	0.44	0.014	0.029	½ Cup.
47096	10	1	do	36 600*	56 360*	27.5	60.2*	0.17	0.44	0.014	0.029	½ Cup.
47096	10	1	do	36 200*	55 500*	32.0*	59.7	0.17	0.44	0.014	0.029	Angular.
47096	10	1	do	35 700*	54 650*	30.0	58.8	0.17	0.44	0.014	0.029	½ Cup.
47096	10	1	9 x ¼	38 780	54 640*	31.2*	56.5	0.17	0.44	0.014	0.029	½ Cup.
47096	10	1	do	26 100*	52 800*	31.2*	46.0*	0.17	0.44	0.014	0.029	½ Cup
42171	10	1	2 x 2 x ¼	41 140*	61 140	27.5	60.7*	0.20	0.44	0.010	0.028	½ Cup.
42171	10	1	do	40 580*	60 940	27.5	59.8	0.20	0.44	0.010	0.028	Angular.
42171	10	1	do	41 110*	60 940	29.8	59.8	0.20	0.44	0.010	0.028	½ Cup.
42171	10	1	do	40 570*	59 860	27.5	62.4*	0.20	0.44	0.010	0.028	Cup.
42171	10	1	do	36 960*	58 930	27.0	57.7	0.20	0.44	0.010	0.028	Angular.
42171	10	1	do	36 450*	58 800	29.5	58.7	0.20	0.44	0.010	0.028	½ Cup.
7463	10a	2	5¼ x 7/16	37 010	62 320*	26.7	55.3	0.20	0.46	0.019	0.034	½ Cup.
4038	10a	2	2 x 2 x 7/16	38 280	61 260	27.5	56.9	0.20	0.47	0.014	0.039	Cup
41209	10a	2	9 x 7/16	37 580	61 450	28.7	52.1	0.22	0.44	0.027	0.036	½ Cup.

* Not within limits of specification.

Not finding an explanation of the falling off in strength of the heavy material in the record of mill specimen tests, the Bureau of Standards then took some of the long columns, which had already been tested, and cut them into lengths to give a slenderness ratio of $\frac{l}{r} = 20$.

These short columns showed that the unit ultimate strength of the heavy columns was considerably more than that of the light ones. Both series indicated a fairly definite point at which permanent set occurred, and showed that this point was lower for the heavy columns than for the light ones, indicating that the increased ultimate strength of the heavy ones came about from the block action of the short, heavy material, and that the elastic limits or yield points are the true indicators of the strength of the two different thicknesses of material. See Fig. 5, from the 1916 Progress Report.

Tests of short columns having a slenderness ratio of $\frac{l}{r} = 20$, made of material which had not been previously stressed, confirmed these results and the belief that the strength of columns is governed, not by the ultimate tensile strength of the material, but by the point at which there is marked departure from an elastic condition. See Fig. 6, from the 1917 Progress Report.

To investigate the question of this critical point, the Bureau of Standards proposed some supplementary specimen tension and compression tests, to be made from pieces, 5 ft. in length, which had been provided from each melt at the time the material was rolled. These 5-ft. pieces were in the nature of coupon material from the columns, and had not been subjected to stresses in the testing of the columns.

Tables 11 to 15 show the record of the supplementary specimen tests made on the coupon material ordered from the same melts as the material for the full-size columns, and Table 22 is the summary of the averages of Table 10 and Tables 11 to 15. When we compare the results shown by Tables 11 to 15 with the mill specimen tensile tests made from these same melts, Table 10, and both with the table of full-size column tests, it will be seen that the ultimate tensile specimen strength is not a correct indicator of the strength of the columns. The ultimate strengths shown on the supplementary specimen tests are very close to the ultimate strengths given for the specimen mill tests, and neither the ultimate strengths from the supplementary tests nor from the mill tests indicate the falling off in strength of the thicker material. It is also evident that the yield point, as recorded by the ordinary commercial tensile specimen tests, even when the machine is run at comparatively slow speeds, as was done in the Pittsburgh Laboratory of the Bureau of Standards, does not give the correct index of the

TABLE 11.

ABSTRACT OF RECORD OF SPECIMEN TESTS.
MADE BY BUREAU OF STANDARDS.

Type	Section.	Specimen no.	Specimen cut from	Ultimate Strength.	Useful Limit Point.
		TENSION TESTS.			
1	Light Sections.	11	5" leg of 5"x3"x5/16" L	57100	35100
		111	"	53900	33200
		11	Root of 5"x3"x5/16" L	65800	33500
		111	"	64000	33200
	4 Ls 5"x3"x5/16" = 9.60□"	11	3" leg of 5"x3"x5/16" L	59000	32700
	1 Pl. 6"x5/16" = 1.875				
	Total = 11.475□"	21	6"x5/16" Pl.	58400	33000
		22	"	58600	33000
		112	"	54900	30800
		COMPRESSION TESTS.			
		Specimen no.	Specimen	l/r of Specimen	Useful Limit Point.
		B18	5"x3"x5/16" L	12.3	34800
		12	"	12.7	33400
		21	6"x5/16" Pl.	—	31000
		21	"	—	31000
		112	"	—	27300
1A	Heavy Sections.	**TENSION TESTS.**			
		Specimen no.	Specimen cut from	Ultimate Strength.	Useful Limit Point
		113	5" leg of 5"x3"x5/8" L	56700	28800
		60	"	55600	30800
	4 Ls 5"x3"x5/8" = 18.44□"	**COMPRESSION TESTS.**			
	1 Pl 6"x5/8" = 3.75	Specimen no	Specimen	l/r of Specimen	Useful Limit Point.
	Total = 22.19□"	1	5"x3"x5/8" L	12.6	28600
		2	"	12.9	25000
		A	"	12.7	27300
1B	Extra Heavy Sections.	**TENSION TESTS.**			
		Specimen no.	Specimen cut from	Ultimate Strength	Useful Limit Point.
	4 Ls 5"x3"x13/16" = 23.36□"				
	1 Pl. 6"x7/8" = 5.25	109	Leg of 5"x3"x13/16" L	57000	31100
	Total = 28.61□"				

TABLE 12.

ABSTRACT OF RECORD OF SPECIMEN TESTS.
MADE BY BUREAU OF STANDARDS.

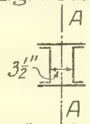
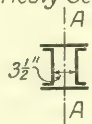
Type	Section	Specimen no.	Specimen cut from	Ultimate Strength.	Useful Limit Point.
		TENSION TESTS.			
2	Light Sections.  2-6" [s 10.5-lb = 6.18□" 2 Pl.s 8"×¼" = 4.00 Total = 10.18□"	27	8"×¼" Pl.	57700	32700
		17	"	56000	29700
		116	"	59600	32200
		44	Root of 6" [10.5-lb.	59700	28400
		115	"	63800	31800
		37	Web of 6" [10.5-lb.	56700	35800
		44	"	56600	33000
		115(22A)	"	60500	37000
		COMPRESSION TESTS.			
		Specimen no.	Specimen	l/r of Specimen	Useful Limit Point.
		37	6" [10.5-lb	19.0	33300
		44	"	"	35000
		115	"	"	33900
		17	8"×¼" Pl.	—	30400
		27	"	—	22800
		116	"	—	31500
		TENSION TESTS.			
		Specimen no.	Specimen cut from	Ultimate Strength.	Useful Limit Point.
2A.	Heavy Sections.  2-6" [s 15.5-lb = 9.12□" 2 Pl.s 8"×½" = 8.00 Total = 17.12□"	118	8"×½" Pl.	58400	31000
		69	"	61500	35000
		66	Web of 6" [15.5-lb.	53500	23500
		117	"	58900	32100
		66	Root of 6" [15.5-lb.	57300	30000
		117	"	60900	29800

TABLE 13.

ABSTRACT OF RECORD OF SPECIMEN TESTS. MADE BY BUREAU OF STANDARDS.

Type	Section.	Tension Tests.			
		Specimen no.	Specimen cut from	Ultimate Strength	Useful Limit Point
	Light Sections	38	Web of 5" [6.5-lb.	54200	36500
		43	"	55500	39700
		38	Root of 5" [6.5-lb.	55700	31000
		43	"	56400	29100
3	2-5" [s 6.5-lb. = 3.90 □"	Compression Tests.			
	2 Pls. 9½" × ¼" = 4.75 Total = 8.65 □"	Specimen no.	Specimen	l/r of Specimen	Useful Limit Point
		38	5" [6.5-lb.	20.2	35200
	Light Sections.	Tension Tests.			
		Specimen no.	Specimen cut from	Ultimate Strength.	Useful Limit Point
4		42.2	Web of 8" [11.25-lb.	57700	36000
	2-8" [s 11.25-lb. = 6.70 □"	42.3	"	60000	40800
	1-8" I 18.0-lb. = 5.33	48	"	57400	37900
	Total = 12.03 □"				
		42	Root of 8" [11.25-lb.	59800	32900
		48	"	61600	33200
		119	"	60000	27600
		50	Flange of 8" I 18.0-lb.	56100	34500
		50	"	56300	35600
		50	Root of 8" I 18.0-lb.	58000	28900
		55	"	57600	31000
		120	"	58900	27200
		55	Web of 8" I 18.0-lb.	55000	36500
		55	"	56000	37800
		Compression Tests			
		Specimen no.	Specimen	l/r of Specimen.	Useful Limit Point
		119	8" [11.25-lb.	16.0	27500
		G	"	15.9	40000
		H	"	16.0	40700
		120	8" I 18.0-lb.	11.8	29000
		F	"	12.0	32000
		E	"	11.9	31600

TABLE 14.

ABSTRACT OF RECORD OF SPECIMEN TESTS. MADE BY BUREAU OF STANDARDS.

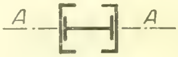
Type	Section.	Tension Tests.			
		Specimen no.	Specimen cut from	Ultimate Strength	Useful Limit Point
4A	Heavy Sections.	80	Web of 8″[18.75-lb	55600	23500
		483-32	"	60200	23700
		79	Root of 8″[18.75-lb	59900	27000
		80	"	61200	23000
		121	"	60100	30200
	2-8″[s 18.75-lb.= 11.02□″	483-32	"	63400	29200
	1-8″I 20.50-lb.= 6.03				
	Total = 17.05□″	81 W	Web of 8″I 20.5-lb.	58000	30000
		82	"	57300	28000
		83	"	57700	27500
		122	"	56100	31300
		81	Root of 8″I 20.5-lb.	62700	26600
		122	"	61200	25100
		83	"	62500	27300
		Compression Tests.			
		Specimen no.	Specimen	l/r of Specimen	Useful Limit Point
		79	8″[18.75-lb.	16.8	26000
		80	"	16.6	26000
		I	"	16.7	28500
		121	"	16.6	34000
		83	8″I 20.5-lb	12.0	28700
		122	"	12.3	29200
		B	"	12.2	25700
		D	"	12.3	30000

TABLE 15.

ABSTRACT OF RECORD OF SPECIMEN TESTS.

MADE BY BUREAU OF STANDARDS.

Type	Section	Tension Tests.			
		Specimen no.	Specimen cut from	Ultimate Strength	Useful Limit Point
5	Light Sections.  8"H Col. 32.0-lb. = 9.17□"	19BX	Flange of 8"H Col. 32.0-lb.	58900	25600
		19BXC	Web of 8"H Col. 32-lb.	58900	37000
		19BX	Root of 8"H Col. 32-lb.	64000	38000
5A	Heavy Sections.  8"H Col. 62.0-lb. = 18.27□"	15CX(C)	Web of 8"H Col. 62.0-lb.	57500	32500
		15CX	Flange of 8"H Col. 62.0-lb.	58100	33800
		15CX(M)	Root of 8"H Col. 62.0-lb.	58000	33700
5B	Extra Heavy Sections.  8"H Col. 91.0-lb. = 26.64□"	24AXC	Web of 8"H Col. 91.0-lb.	56500	25000
		24BXC	"	56500	23100
		24CXC	"	57500	26600
		24AXM	Middle of Flange of 8"H Col. 91.0-lb.	56100	18900
		24CXM	"	56300	26400
		24CX	"	56300	26000
		24BXM	"	56400	20900
		24AX	End of Flange of 8"H Col. 91.0-lb.	56400	17600
		24CX	"	55100	23300
		24AX	"	56700	18500
		24BX	"	56700	21000
		24BX	"	56800	24500
7	Light Sections.  4-5" Bulb Ls 10.1-lb. = 11.88□" 1 Pl. 6"x5/16" = 1.88 Total = 13.76□"	2	Root of 5" Bulb L 10.1-lb.	61600	33900
		6	Bulb of 5" Bulb L 10.1-lb.	61700	33500
		9	"	61500	33000
		Compression Tests.			
		Specimen no.	Specimen	l/r of Specimen	Useful Limit Point
		—	5" Bulb L 10.1-lb.	17.6	34000
		6	"	17.8	34000
		9	"	17.7	35000

strength of the material. It appears necessary, therefore, in order to predict the strength of a column, to determine the nature of the metal by some other means than those generally used.

In the Progress Report of 1917, the Committee mentioned the discussion held by the American Society for Testing Materials at its Annual Meeting, June, 1916, on the relation between proportional limit, elastic limit, and yield point. For the purpose of studying the column tests, the Committee gave careful consideration to this discussion to find whether it was possible to determine some point which, for practical purposes, might be easily located, clearly defined, and at the same time represent the limit where the metal ceases to have structural value. None of the terms defined by the discussions of the American Society for Testing Materials has appealed to the Committee as having these qualities. In searching for a more satisfactory definition, the Committee considered a modification of the suggestion made some years ago by the late J. B. Johnson, M. Am. Soc. C. E. The Committee has defined the critical point as the point which is determined graphically by drawing a line tangent to the envelope of the stress-strain curve, having a slope of one-half that of the last run-up line for its straight, or nearly straight, portion.

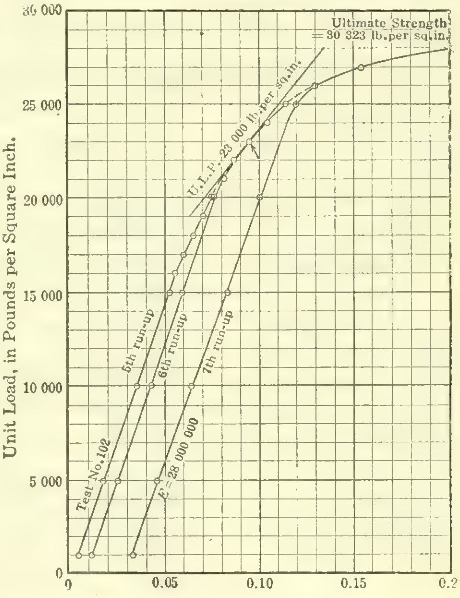


FIG. 7.

Fig. 7 illustrates this method of determination. So as not to confuse this with former definitions of yield point or elastic limit, the Committee has adopted a new term, and calls this the Useful Limit Point, or U. L. P.

In straining a column, there is a point beyond which its structural value is uncertain, and consequently unsafe to rely upon. This point lies somewhere above the region of perfect elasticity and well below the place where manifest yielding occurs. For the study of column tests, the U. L. P. as above defined seems to fulfill these conditions satisfactorily. Careful observations and plotting of the stress-strain curve locate it without chance for controversy. It will be noted that the method is applicable to both tension and compression tests.

TABLE 16.

ABSTRACT OF RECORD OF COLUMN TESTS.
USEFUL LIMIT POINTS FOR TYPES 1, 1A, 1B & 7.

Type	Test no.	Section	Actual Area	Radius of Gyration	Length	$\frac{l}{r}$	Unit Ultimate Strength	Average Ultimate Strength	Unit Useful Limit Point	Average Useful Limit Point
	8	Light Sections.	11.13	2.24	9′-4″	50	32400		28300	
	38		11.17	″	″	″	32800	32700	29200	28800
	77		11.13	″	″	″	33000		29000	
	18		11.49	″	15′-10 7/16″	85	32100		26000	
1	84		11.49	″	″	″	31400	31200	30000	27800
	85	4Ls 5″x3″x5/16″ = 9.60□″	11.35	″	″	″	30000		27500	
		1Pl. 6″x5/16″ = 1.875								
	30	Total = 11.475□″	11.20	″	22′-4 13/16″	120	28600		25400	
	56		11.14	″	″	″	27600	28300	26300	26600
	69		11.30	″	″	″	28700		28000	
	93	Heavy Sections.	22.25	2.36	9′-10″	50	29000		25000	
	96		22.13	″	″	″	28900	29200	24000	24700
	97		22.28	″	″	″	29700		25000	
	91		22.13	″	16′-8 5/8″	85	28000		25700	
1A	94		21.92	″	″	″	28200	28100	24700	25300
	95	4Ls 5″x3″x5/8″ = 18.44□″	22.05	″	″	″	28000		25500	
		1Pl. 6″x5/8″ = 3.75								
	92	Total = 22.19□″	22.15	″	23′-7 3/16″	120	25000		23000	
	98		22.10	″	″	″	25500	25400	23500	23300
	99		22.10	″	″	″	25800		23300	
		Extra Heavy Sections.								
	229		28.43	2.29	16′-2 5/8″	85	28200		26500	
1B	230		28.37	″	″	″	27000	27700	26000	26200
	231	4Ls 5″x3″x13/16″ = 23.36□″	28.53	″	″	″	28000		26000	
		1Pl. 6″x7/8″ = 5.25								
		Total = 28.61□″								
	5	Light Sections.	13.33	2.48	10′-4″	50	34300		30300	
	37		13.30	″	″	″	32700	33400	30600	30600
	76		13.33	″	″	″	33200		31000	
	15		13.40	″	17′-6 13/16″	85	34000		30800	
7*	89		13.43	″	″	″	30000	31600	29000	29400
	90	4-5″ Bulb Ls x 10.1-lb. = 11.88□″	13.29	″	″	″	30700		28500	
		1Pl. 6″x5/16″ = 1.88								
	28	Total = 13.76□″	13.36	″	24′-9 5/8″	120	28300		28400	
	48		13.34	″	″	″	26900	28100	25900	26600
	51		13.33	″	″	″	29000		25600	

* Shapes for Type 7A (Heavy Sections) cannot be obtained

TABLE 17.

ABSTRACT OF RECORD OF COLUMN TESTS.

USEFUL LIMIT POINTS FOR TYPES 2 & 2A—3 & 3A.

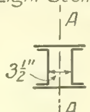
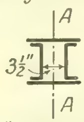
Type	Test no	Section.	Actual Area	Radius of Gyration	Length	$\frac{l}{r}$	Unit Ultimate Strength	Average Ultimate Strength	Unit Useful Limit Point	Average Useful Limit Point
2	11	Light Sections.	10.06	2.31	9′-7½″	50	33100		27400	
	41		10.15	"	"	"	34000	33200	28200	27500
	43		10.06	"	"	"	32400		26900	
	21		9.92	"	16′-4⅜″	85	33700		29200	
	59		9.87	"	"	"	32300	32600	28400	28700
	65	2-6″ [s x 10.5-lb = 6.18□″	10.05	"	"	"	31700		28600	
		2 Pl^s 8″x¼″ = 4.00								
	32	Total = 10.18□″	10.01	"	23′-1 3/16″	120	28300		25600	
	54		10.09	"	"	"	28700	29300	26300	27000
	70		10.07	"	"	"	30900		29000	
2A	101	Heavy Sections.	17.07	2.33	9′-8½″	50	32200		26000	
	124		17.07	"	"	"	32500	32300	27000	26800
	165		16.96	"	"	"	32300		27500	
	145		17.07	"	16′-6 1/16″	85	30700		27000	
	146		17.02	"	"	"	30000	30600	26000	26700
	151	2-6″ [s x 15.5-lb = 9.12□″	17.11	"	"	"	31000		27000	
		2 Pl^s 8″x½″ = 8.00								
	135	Total = 17.12□″	17.04	"	23′-3⅝″	120	28500		26500	
	136		17.02	"	"	"	28800	28100	26500	26300
	137		17.08	"	"	"	27000		26000	
3	1	Light Sections.	8.71	2.34	9′-9″	50	33800		28300	
	2		8.80	"	"	"	34300	34100	28700	28300
	42		8.71	"	"	"	34300		27900	
	14		8.71	"	16′-6⅞″	85	32500		26900	
	87		8.69	"	"	"	31900	32400	29000	28000
	88	2-5″ [s x 6.5-lb = 3.90□″	8.60	"	"	"	32900		28000	
		2 Pl^s 9½″x¼″ = 4.75								
	24	Total = 8.65□″	8.79	"	23′-4 13/16″	120	30600		27900	
	35		8.70	"	"	"	30200	30300	28600	28000
	47		8.74	"	"	"	30000		27500	
3A	105	Heavy Sections.	16.44	2.39	9′-11½″	50	29800		25500	
	168		16.45	"	"	"	29300	29500	25000	25300
	171		16.39	"	"	"	29500		25500	
	147		16.41	"	16′-11⅛″	85	28000		26000	
	148		16.39	"	"	"	28000	28000	25500	25700
	150	2-5″ [s x 11.5-lb = 6.76□″	16.37	"	"	"	28000		25500	
		2 Pl^s 9½″x½″ = 9.50								
	138	Total = 16.26□″	16.50	"	23′-10 13/16″	120	28000		26500	
	139		16.45	"	"	"	26100	26900	26000	25700
	143		16.42	"	"	"	26600		24500	

TABLE 18.

ABSTRACT OF RECORD OF COLUMN TESTS.

USEFUL LIMIT POINTS FOR TYPES 4&4A–8&8A.

Type	Test no.	Section.	Actual Area	Radius of Gyration	Length.	$\frac{l}{r}$	Unit Ultimate Strength	Average Ultimate Strength.	Unit Useful Limit Point.	Average Useful Limit Point.
	6	Light Sections.	11.69	2.38	9'-11"	50	35700		29900	
	36		11.69	"	"	"	36500	36900	33000	32000
	75		11.69	"	"	"	38600		33000	
	13		11.66	"	16'-10 5/16"	85	34000		30400	
4	79		11.73	"	"	"	33900	34000	31500	31100
	80	2-8" [s x 11.25-lb. = 6.70 □"	11.64	"	"	"	34000		31500	
		1-8" I x 18.00-lb. = 5.33								
	26	Total = 12.03 □"	11.69	"	23'-9 5/8"	120	32000		30300	
	45		11.61	"	"	"	30700	31900	28200	30000
	49		11.68	"	"	"	33100		31600	
	102	Heavy Sections.	16.98	2.32	9'-8"	50	30300		23000	
	166		16.84	"	"	"	29000	29100	22500	22500
	169		16.81	"	"	"	28000		22000	
	152		16.86	"	16'-5 3/16"	85	26000		22500	
4A	153		16.89	"	"	"	26400	26600	23500	23300
	154	2-8" [s x 18.75-lb. = 11.02 □"	17.00	"	"	"	27500		24000	
		1-8" I x 20.50-lb. = 6.03								
	132	Total = 17.05 □"	16.98	"	23'-2 7/16"	120	23500		21500	
	133		17.07	"	"	"	23900	23900	22000	21700
	134		16.96	"	"	"	24200		21500	
	9	Light Sections.	10.98	2.46	10'-3"	50	35900		30900	
	39		11.01	"	"	"	35100	35700	30500	31300
	78		11.03	"	"	"	36200		32500	
	16		11.09	"	17'-5 1/8"	85	31400		31400	
8	81		11.05	"	"	"	32900	32800	30000	31000
	82	4-4" Z's x 1/4" = 9.64 □"	11.11	"	"	"	34000		31500	
		1 Pl. 7" x 1/4" = 1.75								
	27	Total = 11.39 □"	11.06	"	24'-7 3/16"	120	29100		28200	
	46		11.11	"	"	"	29400	29700	25900	28200
	50		11.16	"	"	"	30400		30400	
	100	Heavy Sections.	25.78	2.51	10'-5 1/2"	50	33000		29000	
	126		25.86	"	"	"	33500	32900	29500	29000
	127		25.83	"	"	"	32200		28500	
	156		25.56	"	17'-9 3/8"	85	31300		31000	
8A	160		25.57	"	"	"	25300	31400*	20000	29700*
	161	4-4" Z's x 5/8" = 22.20 □"	25.43	"	"	"	31500		28500	
		1 Pl. 7" x 5/8" = 4.38								
	162	Total = 26.58 □"	25.66	"	25'-1 3/16"	120	27500		24500	
	163		25.70	"	"	"	27000	27300	26000	25500
	164		25.68	"	"	"	27500		26000	

* Test no. 160 omitted in determining the averages.

TABLE 19.

ABSTRACT OF RECORD OF COLUMN TESTS.

USEFUL LIMIT POINTS FOR TYPES 5, 5A & 5B.

Type	Test No.	Section.	Actual Area	Radius of Gyration	Length	$\frac{l}{r}$	Unit Ultimate Strength.	Average Ultimate Strength.	Unit Useful Limit Point.	Average Useful Limit Point.
	107	Light Sections.	9.62	1.98	8′-3″	50	33000		35000	
	122		9.65	"	"	"	38000	38000	34500	34700
	123	A A	9.65	"	"	"	38000		34500	
	114		8.61	"	14′-0 5/16″	85	36000		34000	
5	116		9.66	"	"	"	34000	34300	32500	32800
	119	8″-H Col. x 32-lb.	9.59	"	"	"	33000		32000	
		= 9.17□″								
	110		8.64	"	19′-9 5/8″	120	33900		33000	
	111		8.68	"	"	"	31000	32000	30200	31100
	108		8.61	"	"	"	31000		30000	
	106	Heavy Sections.	17.86	2.09	8′-8½″	50	34000		30000	
	120		17.82	"	"	"	36100	35400	31000	31200
	121	A A	17.86	"	"	"	36000		32500	
	115		18.05	"	14′-9 5/8″	85	33900		32000	
5A	117		17.73	"	"	"	32000	32300	30500	30600
	118	8″-H Col. x 62-lb.	17.77	"	"	"	31000		29300	
		= 18.27□″								
	109		17.76	"	20′-10 13/16″	120	30000		29000	
	112		17.97	"	"	"	30000	30000	30000	29300
	113		17.73	"	"	"	30000		29000	
	225	Extra Heavy Sections.	27.49	2.17	9′-0½″	50	24000		19000	
	226		27.18	"	"	"	24600	25200	21100	19700
	182	A A	27.50	"	"	"	27100		19000	
	227		27.28	"	15′-4 7/16″	85	22600		19600	
5B	228		27.27	"	"	"	23000	23500	20000	20200
	183	8″-H Col. x 91-lb.	27.45	"	"	"	24800		21000	
		= 26.64□″								
	223		27.50	"	21′-8 3/8″	120	21000		19000	
	224		27.54	"	"	"	21000	21300	18500	19200
	181		27.57	"	"	"	21800		20000	

TABLE 20.

ABSTRACT OF RECORD OF COLUMN TESTS.

USEFUL LIMIT POINTS FOR TYPES 6 & 6A – 10 & 10A.

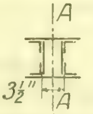
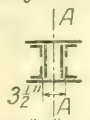
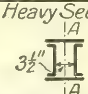
Type	Test no.	Section.	Actual Area	Radius of Gyration	Length	$\frac{l}{r}$	Unit Ultimate Strength	Average Ultimate Strength	Unit Useful Limit Point.	Average Useful Limit Point.
6	10	Light Sections.	13.72	2.31	9'-7½"	50	31200		24200	
	40	A	13.78	"	"	"	31500	31600	23800	24200
	44		13.72	"	"	"	32200		24600	
	22		13.65	"	16'-4⅜"	85	27800		23700	
	63	A	13.68	"	"	"	29500	29100	24500	24800
	64	1-I 10"×25-lb.=7.37□"	13.60	"	"	"	30100		26200	
		2 Pl.s 11"×5/16"=6.88								
	31	Total = 14.25□"	13.73	"	23'-1 3/16"	120	26700		24100	
	52		13.75	"	"	"	27700	27200	24800	24500
	71		13.65	"	"	"	27300		24500	
6A.	103	Heavy Sections.	23.98	2.47	10'-3½"	50	31200		21000	
	125	A	23.75	"	"	"	32800	32100	22500	21800
	128		23.72	"	"	"	32400		22000	
	155		23.69	"	17'-5 15/16"	85	27200		23500	
	158	A	23.73	"	"	"	26800	26800	23000	23300
	159	1-I 10"×35-lb.=10.29□"	23.72	"	"	"	26400		23500	
		2 Pl.s 11"×⅝"=13.75								
	129	Total = 24.04□"	23.64	"	24'-8⅜"	120	24900		22000	
	130		23.83	"	"	"	24000	24800	23000	22500
	131		23.76	"	"	"	25500		22500	
10	3	Light Sections.	10.77	2.33	9'-8½"	50	36100		26000	
	4	A	10.80	"	"	"	35700	35800	27700	26800
	53		10.86	"	"	"	35600		26800	
	19	3½"	10.78	"	16'-6 1/16"	85	32500		26600	
	83	A	10.76	"	"	"	31900	32100	27000	26900
	86	2 Pl.s 9"×¼"=4.500□"	10.76	"	"	"	31900		27000	
		4 Ls 2"×2"×¼"=3.760								
	23	2 Pl.s 5¼"×¼"=2.625	10.75	"	23'-3⅝"	120	28400		27000	
	33	Total =10.885□"	10.76	"	"	"	28300	28400	25800	26500
	34		10.75	"	"	"	28400		26700	
10A.	104	Heavy Sections.	18.60	2.34	9'-9"	50	31400		25500	
	167	A	18.52	"	"	"	32000	31800	26000	26000
	170		18.46	"	"	"	32100		26500	
	144	3½"	18.67	"	16'-6⅞"	85	28000		24500	
	149	A	18.65	"	"	"	28000	28300	24500	24500
	157	2 Pl.s 9"×7/16"= 7.88□"	18.69	"	"	"	29000		24500	
		4 Ls 2"×2"×7/16"= 6.24								
	140	2 Pl.s 5¼"×7/16"=4.59	18.64	"	23'-4 13/16"	120	27000		24500	
	141	Total =18.71□"	18.80	"	"	"	25900	26300	23500	24000
	142		18.65	"	"	"	26000		24000	

TABLE 21.

ABSTRACT OF RECORD OF COLUMN TESTS.

USEFUL LIMIT POINTS FOR TYPES 1 & 1A – 2 & 2A – 4 & 4A.

Slenderness Ratio $\frac{l}{r}$ = 155.

Type	Test no.	Section.	Actual Area	Radius of Gyration	Length.	$\frac{l}{r}$	Unit Ultimate Strength	Average Ultimate Strength	Unit Useful Limit Point.	Average Useful Limit Point.
		Light Sections. 								
	207		11.84	2.24	28'-11$\frac{3}{16}$"	155	27000		26000	
1	218		11.69	"	"	"	25600	26200	25000	25200
	222	4 Ls 5"x3"x$\frac{5}{16}$" = 9.60 □"	11.68	"	"	"	26000		24500	
		1 Pl. 6"x$\frac{5}{16}$" = 1.875								
		Total = 11.475 □"								
		Heavy Sections. 								
	205		22.35	2.36	30'-5$\frac{13}{16}$"	155	22200		21300	
1A	206		22.35	"	"	"	23200	22700	22400	21900
	208	4 Ls 5"x3"x$\frac{5}{8}$" = 18.44 □"	22.31	"	"	"	22700		21000	
		1 Pl. 6"x$\frac{5}{8}$" = 3.75								
		Total = 22.19 □"								
		Light Sections. 								
	215		10.39	2.31	29'-10$\frac{1}{16}$"	155	26100		25700	
2	217		10.42	"	"	"	27800	26500	26800	25700
	221	2-6" [s x 10.5-lb = 6.18 □"	10.46	"	"	"	25600		24700	
		2 Pls 8"x$\frac{1}{4}$" = 4.00								
		Total = 10.18 □"								
		Heavy Sections. 								
	214		16.75	2.33	30'-1$\frac{1}{8}$"	155	24700		24000	
2A	216		16.87	"	"	"	25000	24900	23000	24000
	219	2-6" [s x 15.5-lb = 9.12 □"	16.81	"	"	"	25000		25000	
		2 Pls 8"x$\frac{1}{2}$" = 8.00								
		Total = 17.12 □"								
		Light Sections 								
	211		11.81	2.38	30'-8$\frac{7}{8}$"	155	24000		21000	
4	212		11.93	"	"	"	23200	23600	23000	22300
	220	2-8" [s x 11.25-lb = 6.70 □"	11.84	"	"	"	23500		23000	
		1-8" I x 18.00-lb = 5.33								
		Total = 12.03 □"								
		Heavy Sections. 								
	209		16.74	2.32	29'-11$\frac{5}{8}$"	155	24000		24000	
4A	210		16.67	"	"	"	23000	23300	23000	23300
	213	2-8" [s x 18.75-lb = 11.02 □"	16.33	"	"	"	23000		23000	
		1-8" I x 20.50-lb = 6.03								
		Total = 17.05 □"								

The Committee has carefully considered the application of this method to the stress-strain diagrams of full-size columns. Tables 16 to 21 give the U. L. P. (Useful Limit Points) for the column tests, together with the average for the three of each series, also the ultimate strength, so that comparison may be made between the values of the U. L. P. and the ultimate strengths, and Plates LVIII, LIX, and LX are given as samples to illustrate the method used by the Committee in determining the U. L. P. values. To publish all the diagrams used in these investigations would cost the Society a large sum of money, and the Committee is of the opinion that such publication in this report is not necessary. At the completion of the work of the Committee, the tracings of the diagrams will be filed in the Library, and they can be reproduced at a nominal expense for those engineers who desire to make a special study of this phase of the work.

It will be seen from a study of the tables of U. L. P. values that this method offers an extremely valuable indication of the strength of full-size columns. It will be seen, further, that the shape of the cross-section of the columns, as covered by the programme of the Committee, is of minor importance when considered in connection with the strength of the material, as shown by the U. L. P. values.

As mentioned above, and as will be seen from Tables 11 to 15, the specimen coupon tests are not complete. The Bureau of Standards planned to carry on a very elaborate supplementary investigation during 1917. For some time, however, the testing capacity of the Bureau has been given over to testing materials for war purposes. The tests here published show all that Dr. Stratton can promise until the close of the War. The Committee feels that the results shown by these tests would not be changed materially by additional tests in the series, and, realizing the demand for information on the subject, desires to make a final report based on the available data.

The data mentioned above, namely Tables 1 to 21, inclusive, showing the values of the U. L. P. (Useful Limit Points), furnish all the information which the Committee has obtained in accordance with the first item of its commission—to determine the ultimate strength of steel columns and struts. In making this as a report, the Committee has not attempted to correlate these tests with the older ones shown in its report of 1910. Some of the columns given in the 1910 report were tested many years ago, when it was not thought necessary to have such accurate measurements as have been made in the present series. To attempt to correlate the older column tests with the present ones would, in the opinion of the Committee, be of no value except as a general speculation.

It will be seen at once from the study of the foregoing tables that the record is unsatisfactory, on account of the large variation in ultimate strengths and U. L. P. (Useful Limit Points) of the columns. Some

of this variation may be brought about by the shape of the column, but most of it can be explained by the condition of the material. Plate LXI is a summary of the curves of all of the U. L. P. values. A surprising feature of these curves is the variation in U. L. P. of 15 000 lb. per sq. in. between columns of Types 5 and $5B$. The average U. L. P. for $\frac{l}{r} = 50$, for columns of Type 5, light section, is 34 700 lb. per sq. in., and for Type $5B$, extra heavy section, it is 19 700 lb. per sq. in. The lower value is but 57% of the higher, and there is a variation of 28% from the mean. These extreme variations may be abnormal, but it must be remembered that the tests were made on material specially chosen, and supposedly the best that the mills could produce.

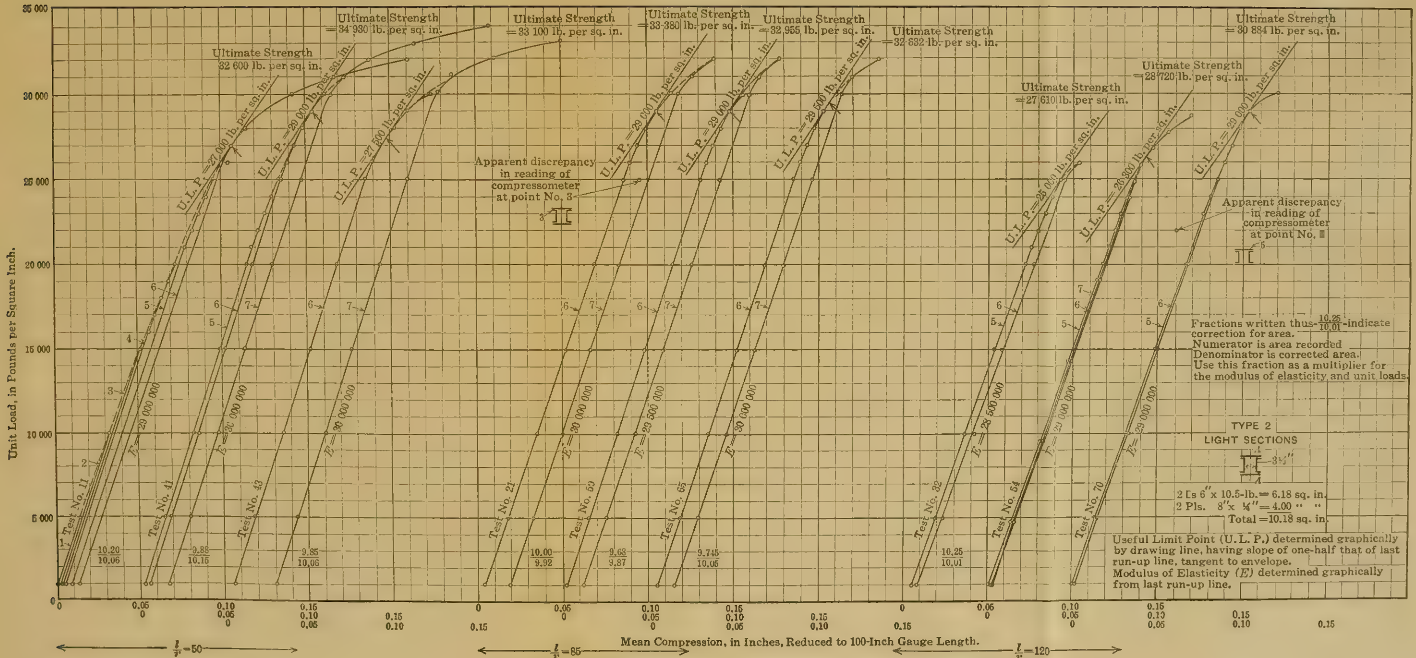
On account of this large range in the results, it is not possible for the Committee to execute satisfactorily the first portion of its commission, which was to determine the ultimate strength of steel columns. The Committee does not believe it is safe to take the average strength of all the columns which the Bureau of Standards has tested as a basis for the design of columns where there is uncertainty as to the quality of the material. In order to make a constructive report, therefore, it is necessary to consider the second part of the Committee's commission, namely, to determine the safe working value for columns, and to make this a safe working value conservative enough to cover the wide range in the results shown by the Committee's programme.

In considering the safe working value for a column, it is necessary to take account of a number of uncertain and sometimes unknown factors. Some of these might be enumerated as follows:

(1) Material:
- (*a*) Quality of steel.—High and low carbon.—Amount of work received.
- (*b*) Uniformity.—Segregations.—Hard and soft spots.
- (*c*) Workmanship.—Initial bends.—Inadequate or poor riveting.

(2) Condition of loading:
- (*a*) End bearing.—Pin or square ends.—Ends not milled perpendicular to axis.
- (*b*) Eccentricity.—Load not applied at axis of column.
- (*c*) Transverse loading.—Combined beam and column action.
- (*d*) Repeated stresses.—Loads repeated and perhaps alternated many times.

(3) Use:
- (*a*) Overloading.—Impossibility of always determining exact loading.
- (*b*) Secondary stresses.— Sometimes a considerable proportion of the primary stress.


Ultimate Strength 32 600 lb. per sq. in.
Ultimate Strength =34 930 lb. per sq. in.
Ultimate Strength =33 100 lb. per sq. in.
Ultimate Strength =33 380 lb. per sq. in.
Ultimate Strength =32 955 lb. per sq. in.
Ultimate Strength =32 632 lb. per sq. in.
Ultimate Strength =27 610 lb. per sq. in.
Ultimate Strength =28 720 lb. per sq. in.
Ultimate Strength =30 884 lb. per sq. in.
U. L. P. =27 000 lb. per sq. in.
U. L. P. =29 000 lb. per sq. in.
U. L. P. =27 500 lb. per sq. in.
U. L. P. =29 000 lb. per sq. in.
U. L. P. =29 000 lb. per sq. in.
U. L. P. =29 500 lb. per sq. in.
U. L. P. =25 000 lb. per sq. in.
U. L. P. =26 300 lb. per sq. in.
U. L. P. =29 000 lb. per sq. in.
Apparent discrepancy in reading of compressometer at point No. 3
Apparent discrepancy in reading of compressometer at point No. 5
E=29 000 000
E=30 000 000
E=30 000 000
E=30 000 000
E=29 500 000
E=30 000 000
E=28 500 000
E=29 000 000
E=29 000 000
Test No. 11
Test No. 41
Test No. 43
Test No. 21
Test No. 59
Test No. 65
Test No. 32
Test No. 54
Test No. 70
10.20/10.06
9.88/10.16
9.85/10.06
10.00/9.92
9.68/9.87
9.745/10.05
10.25/10.01
Fractions written thus 10.25/10.01 indicate correction for area.
Numerator is area recorded
Denominator is corrected area.
Use this fraction as a multiplier for the modulus of elasticity and unit loads.
TYPE 2
LIGHT SECTIONS
3½"
2 [s 6" x 10.5-lb. = 6.18 sq. in.
2 Pls. 8" x ¼" = 4.00 " "
Total = 10.18 sq. in.
Useful Limit Point (U. L. P.) determined graphically by drawing line, having slope of one-half that of last run-up line, tangent to envelope.
Modulus of Elasticity (E) determined graphically from last run-up line.
Unit Load, in Pounds per Square Inch.
Mean Compression, in Inches, Reduced to 100-Inch Gauge Length.
l/r=50
l/r=85
l/r=120

(*c*) Deterioration or corrosion.—Exposure to corrosive elements.

(*d*) Facilities for inspection.—Forms of columns which may be frequently and easily examined.

(*e*) Warning of failure.—Practically no warning, as compared with tension members.

(*f*) Gravity of injury caused by failure.—Damage to life and property in some cases, or only temporary inconvenience in other cases.

(*g*) Cost of replacement.

In the early days of the iron and steel industry, it was the custom of engineers to adopt a working unit stress for tension members of one-quarter of the unit ultimate specimen tensile strength, and they spoke of a factor of safety of four. When we consider the distortions produced by the stretching of tension members after being strained above the yield point, and that manifest yielding and failure occur in columns when the stress reaches about one-half of the specimen ultimate strength in tension, it is evident that the factor of safety obtained by this older method was nearer two than four.

In a structure having both tension and compression members, the desideratum in determining a factor of safety is to obtain a working stress so that all parts of the structure have an equal capacity to resist the applied loadings.

Your Committee has made no original investigations of the strength of full-size riveted tension members, and, therefore, cannot make as definite a comparison with full-size riveted columns as would be desirable. It may be stated, however, that the usual working stress in tension is approximately one-half the elastic value of the metal, and the Committee assumes that, in view of all the factors mentioned above, columns should have a safety factor of at least two, based on the U. L. P., in order to be on a parity with tension members.

The average U. L. P. of all the column tests in the Committee's programme for slenderness ratios, $\frac{l}{r} = 50$ and $\frac{l}{r} = 85$, is 27 200 lb. per sq. in. The U. L. P. for the extra heavy section, Type 5*B*, slenderness ratio $\frac{l}{r} = 50$, which is the lowest value observed, is 19 700 lb. per sq. in., which is 28% below the average, and this appears to be too wide a margin of under-run for safety. It would seem to be necessary, therefore, in recommending a working stress, to assume a U. L. P. lower than the average of all the tests. If we take as a safe assumption the mean between the lowest value and the average value, this mean will be 23 500 lb., or approximately 24 000 lb. per sq. in. The factor of two applied to 24 000 lb. will give a safe working value

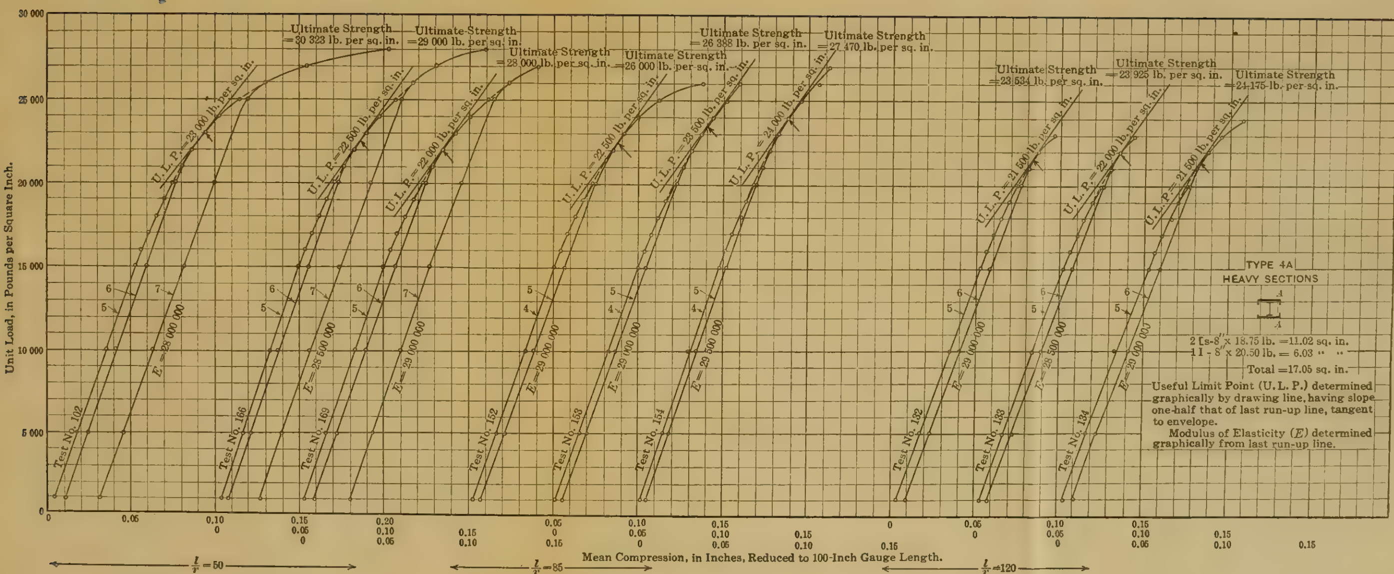
of 12 000 lb. per sq. in. for columns, which stress the Committee would recommend. In this recommendation it is assumed that we are dealing only with static loads, and that a percentage of the static stress will be added to cover the impact due to moving loads.

Lacking further experimental data, the Committee regards it as unwise to assume a higher working stress than 12 000 lb. per sq. in. for columns in which the ordinary grade of structural steel (60 000 lb. ultimate tensile strength desired), is specified. It is, of course, impracticable to know in advance the precise U. L. P., or any other factor of strength, which the metal used may develop. It would be possible to specify the desired value, and, in important structures, to inspect the material with sufficient care to insure the rejection of all which failed to come up to the specification. Later in the report suggestion is made of a method whereby those engineers who have ample facilities for testing materials may adopt such working stresses as are warranted by the special material tests. It must be remembered, however, that a large portion of the material which is classed as structural steel is furnished on the manufacturer's specifications, and, in view of the Committee's findings, probably most structural steel is incompletely tested.

The Committee would recommend that this working stress of 12 000 lb. be used for columns up to a slenderness ratio of $\frac{l}{r} = 80$, and, above this slenderness ratio, the Committee would reduce the working stress to allow for uncertainties. The Committee realizes that the results as given in its programme show that the slenderness ratio has a comparatively small effect, up to values of $\frac{l}{r} = 120$. It must be remembered, however, that the tests were made by the Bureau of Standards under extremely favorable conditions, the ends of the column being scraped so as to give a bearing precisely perpendicular to the axis of the column. The Committee would recommend a working stress of 8 000 lb. per sq. in. for slenderness ratio of $\frac{l}{r} = 120$, and that the working stresses for slenderness ratios between 80 and 120 be determined by interpolation.

The Committee realizes that the working value recommended for short columns is lower than that given by the American Railway Engineering Association formula, which has been in use for a number of years. Originally this formula was $\frac{P}{A} = 16\,000 - 70\,\frac{l}{r}$ and, later, the upper section was truncated to a maximum working stress of 14 000 lb. The Committee feels, in view of its studies in regard to the U. L. P., that there is no warrant for high working stresses in short columns.

PLATE LIX.
PAPERS, AM. SOC. C. E.
DECEMBER, 1917.
FINAL REPORT OF
SPECIAL COMMITTEE ON
COLUMNS AND STRUTS.



Argument might be made that higher working values than those recommended by the Committee have been in use for long periods of years, without failure, but the Engineering Profession does not need to be reminded of accidents which have been caused by the failure of columns. Furthermore, it is evident that the present processes used in the manufacture of steel do not produce a uniform material, and that the present methods of testing do not give a correct indication of the compressive strength of the material.

TABLE 22.—Summary of Results of Mill Tests and Bureau of Standards Tests.

Type.	Mill Specimen Tests made at Pittsburgh.			Bureau of Standards Specimen Tests.			Average U.L.P. of full-size columns $\frac{l}{r}$ = 50 and 85	Average ultimate strength of full-size columns $\frac{l}{r}$ = 50 and 85
	Average drop of beam.	Average ultimate strength	Number of tests.	Average U. L. P.	Average ultimate strength	Number of tests.		
1	34 000	58 900	7	33 100	59 000	8	28 300	32 000
1A	38 300	59 500	7	29 800	56 200	2	25 000	28 600
1B	36 400	59 600	4	31 100	57 000	1	26 200	27 700
2	36 500	57 700	5	32 500	58 800	8	28 100	32 900
2A	36 900	59 700	4	30 200	58 400	6	26 800	31 400
3	39 800	55 600	5	34 100	55 400	4	28 200	33 300
3A	37 900	59 200	2	.....	.....	.....	25 500	28 800
4	36 500	58 100	5	33 800	58 000	13	31 600	35 400
4A	37 700	59 400	4	27 100	59 700	13	22 900	27 900
5	.....	.....	.....	37 500	61 500	2	33 800	36 200
5A	.....	.....	.....	33 300	57 900	3	30 900	33 800
5B	.....	.....	.....	22 600	56 400	12	20 000	24 400
6	34 600	57 600	9	.....	.....	.....	24 500	30 400
6A	38 300	60 400	3	.....	.....	.....	22 600	29 500
7	34 600	59 400	5	33 500	61 600	3	30 000	32 500
8	39 200	56 300	6	.....	.....	.....	31 200	34 200
8A	37 600	58 700	3	.....	.....	.....	29 400	32 300
10	37 600	58 100	16	.....	.....	.....	26 800	34 000
10A	37 600	61 700	3	.....	.....	.....	25 200	30 100

The difference between the strength of the material as indicated by the mill tests made in the ordinary commercial manner and the same material when fabricated into columns is strikingly shown by Table 22. It will be noted from the portion of the table giving the values for the specimens tested at the mill that neither the "yield points", as indicated by the drop of the beam, nor the ultimate strengths, bear any consistent relation to the U. L. P.'s, or ultimate strengths,

of the full-size columns. In most cases, material which gave the lower strength in the full-size columns gave the higher "yield point", as indicated by the mill tests, and, on the other hand, the U. L. P.'s of the specimens, as determined by the Bureau of Standards, bear a very definite and constant relation to the U. L. P.'s of the full-size columns. While the studies of the Committee point to a method whereby the strength of material for columns may be correctly determined, until some such method has been generally adopted, the Committee must recommend a conservative safe working value.

The Committee has not attempted to determine safe working values for columns with slenderness ratio above $\frac{l}{r} = 120$. Some tests have been made and recorded for a slenderness ratio of $\frac{l}{r} = 155$ which may serve as a guide to engineers, but the Committee does not feel that these data are sufficient to carry its recommendation higher than a slenderness ratio of $\frac{l}{r} = 120$.

The Committee's report covers only sections designed to avoid the necessity of latticing or battens, and tested with square ends. It has not been possible to consider the effect of different arrangements of the ends. Details, such as lattice bars, tie-plates, batten-plates, and pin-plates, have not been considered, because the Committee felt that the more important factor was the strength of the material in the columns, and that the question of details could be left until later. The Committee would refer engineers to the reports of the Committee on Iron and Steel Structures of the American Railway Engineering Association, which has worked in co-operation with your Committee, and has considered some of the details mentioned above.

Your Committee's report covers only one grade of steel, which is the ordinary structural grade, with a desired ultimate tensile strength of 60 000 lb. per sq. in. Your Committee has not considered, except in a very general way, the question of transverse loadings, repeated loadings, and long-time tests, and these features will have to await the accumulation of additional data.

The Committee realizes the demand of many engineers for a scientific, rational column formula. Numerous theoretical formulas have been promulgated from time to time, all of them based on entirely elastic material of uniform grade. At the present time it seems useless to the Committee to attempt to write a rational column formula, from a theoretical standpoint, with the possibility of variations of 28% in the strength of the material.

PLATE LX.
PAPERS, AM. SOC. C. E.
DECEMBER, 1917.
FINAL REPORT OF
SPECIAL COMMITTEE ON
COLUMNS AND STRUTS.

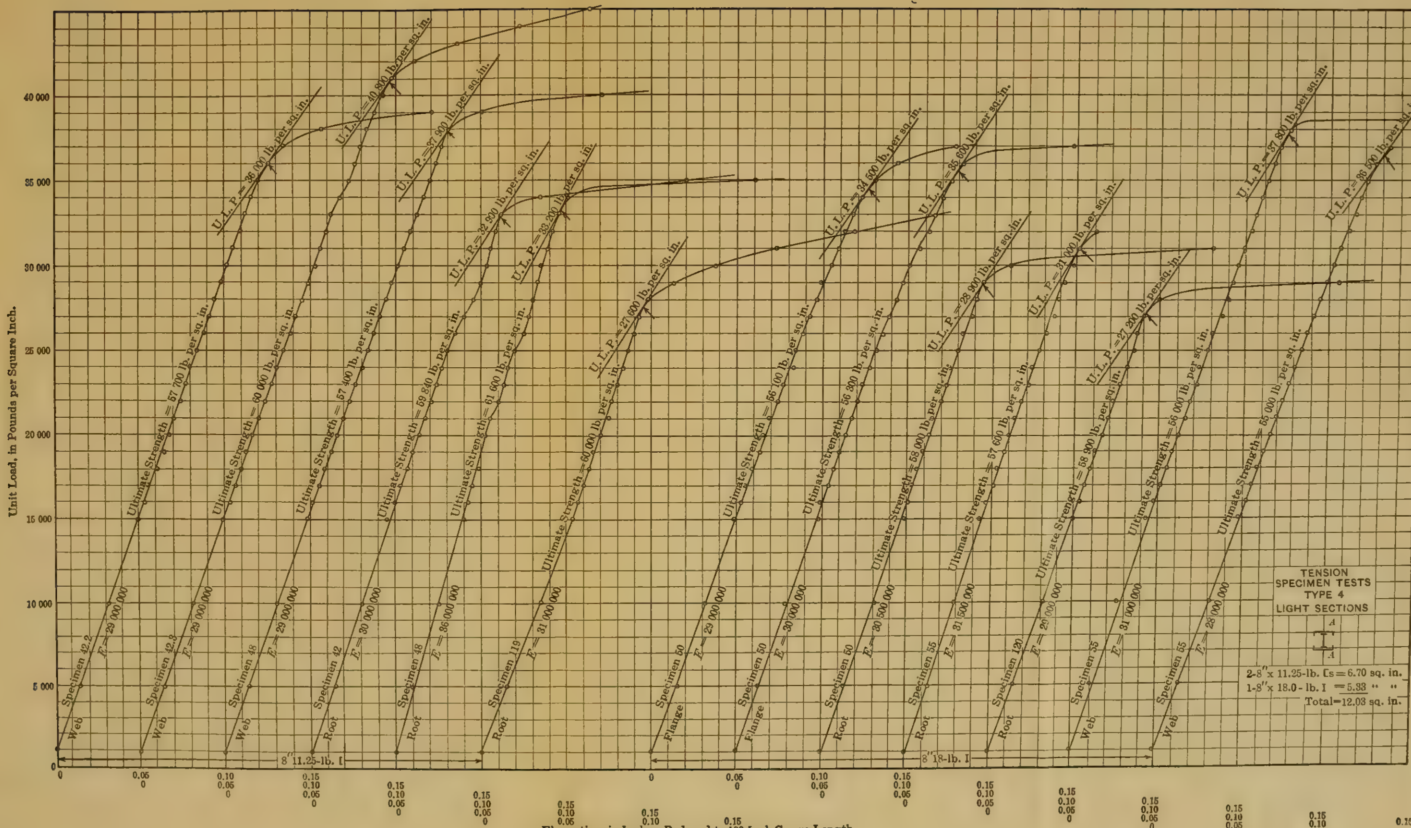


TABLE 23.—Summary of Results, Showing Relation Between Full-Size Column Tests and Specimen Tests.

Type.		U. L. P's of Full-Size Columns Each value is the average of three test results.			Weighted average U. L. P's of all tension and Comp. Spes. tests.	Percentage which U. L. P's of full-size column tests are below U. L. P's of Spec. tests.	Total number of specimen tests.	Average U. L. P's of tension tests on web specimens.	Percentage which U. L. P's of full-size column tests are below U. L. P's of web specimen tests.	Number of web tests on web specimens.
		$\frac{l}{r} = 50.$	$\frac{l}{r} = 85.$	Average of $\frac{l}{r} = 50$ and 85.						
1 1A		28 800 24 700	27 800 25 300	28 300 25 000	33 400 28 400	15.3 12.0	13 5	33 600 29 800	15.8 16.1	5 2
2 2A		27 500 26 800	28 700 26 700	28 100 26 800	32 900 32 100	14.3 16.5	14 6	33 800 32 500	16.9 17.5	6 3 *
3		28 300	28 000	28 200	35 300	20.1	5	38 100	26.0	2
4 4A		32 000 22 500	31 100 23 300	31 600 22 900	36 000 27 200	12.2 15.8	18 21	37 800 25 600	16.4 10.5	5 6
5 5A 5B		34 700 31 200 19 700	32 800 30 600 20 200	33 800 30 900 20 000	37 200 33 400 23 200	9.1 7.5 13.8	3 3 12	37 000 32 500 24 900	8.7 4.9 19.7	1 1 3
7		30 600	29 400	30 000	33 900	11.5	6	33 200	9.6	2
					Averages	13.5			14.7	

* Test No. 66 omitted in taking averages.

The careful selection of specimens located by the inspector, and the testing of these with extensometers, means an increased cost of the material, both to the rolling mill and to the purchaser. The Committee believes, however, that the resultant saving and the assurances of safety for large tonnages and important structures would warrant such increased expense. Possibly such inspection, if extensively carried out, would improve the methods of manufacture of steel, thereby producing a more uniform material.

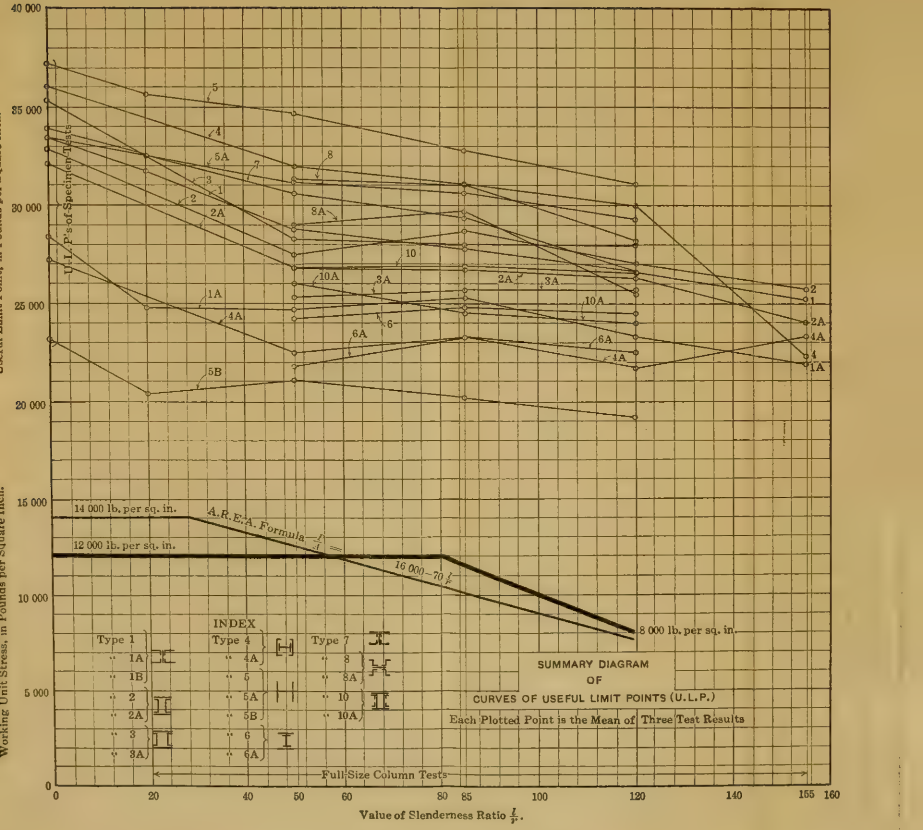
To those engineers who desire to undertake such inspection, the Committee would recommend the study of Table 23, which shows the comparison between the U. L. P.'s of the specimen tests, and the U. L. P.'s of the full-size tests. As before stated, the Bureau of Standards has not been able to supply the Committee with the full quota of specimen tests. It will be seen that, where only a few specimen tests were furnished, the complete record of the material, so far as relates to the condition in the flange and the root, cannot be satisfactorily averaged, but the Committee is giving all the information that it has at hand. Some of the specimen tests were made in tension and some in compression, and these tests from different parts of the cross-sections of the columns were weighted in proportion to arbitrarily assumed areas. The data are too meager to warrant a recommendation as to the use of the average of these percentages in the design of future columns, but they may serve as a guide to those engineers who desire to begin investigations. As most specimen tests are made from the long legs of angles and the webs of I-beams and channels, on account of the convenience in securing the specimens, the table giving the percentage of the U. L. P. of full-size columns below web specimens should afford the best guide as to what might be the probable U. L. P. strength of full-size columns after the specimen U. L. P. was known.

As stated in the beginning of this report, your Committee has been in service for 9 years, and it is the feeling of its members that collateral lines of investigation can be carried out by the appointment of a new Committee, rather than by the continuance of the old one.

Complete records of the work of the Committee will be filed in the Library of the United Engineering Society. At the close of the War, the Bureau of Standards may be in a position to complete the programme of testing the coupon material set aside for specimen tests, and the Committee has the assurance of Dr. Stratton that the record of this result will also be available for the files of the Library.

Your Committee desires to express thanks to the Bureau of Standards, which provided the tests on which this report is based, and to Dr. S. W. Stratton, Director, and his assistants, who made many investigations and suggestions, for their loyal co-operation and support.

PLATE LXI.
PAPERS, AM. SOC. C. E.
DECEMBER, 1917.
FINAL REPORT OF
SPECIAL COMMITTEE ON
COLUMNS AND STRUTS.



Believing that the foregoing report completes the work which the Committee can at present perform to advantage, it now asks to be discharged.

For the Committee,

LEWIS D. RIGHTS,
Chairman.
C. W. HUDSON,
Secretary.

COMMITTEE:
JAMES H. EDWARDS,
C. W. HUDSON,
CHARLES F. LOWETH,
RALPH MODJESKI,
LEWIS D. RIGHTS,
GEORGE F. SWAIN,
EMIL SWENSSON,
J. R. WORCESTER.

NOVEMBER 22D, 1917.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

PROGRESS REPORT OF THE SPECIAL COMMITTEE ON THE REGULATION OF WATER RIGHTS.*

TO THE AMERICAN SOCIETY OF CIVIL ENGINEERS:

Previous Reports.—The Special Committee on a National Water Law presented a unanimous report printed on pages 2747-2751 of the December, 1915, *Proceedings*, pointing out the relative importance of various uses of water, and embodying certain principles which should be generally approved as forming the foundations for a body of water laws. Following the presentation of this report, extra copies were printed and distributed to various organizations of engineers and to bar associations, invitations being given to co-operate with the Special Committee in the discussion of these principles.

The original Committee of nine members, distributed geographically throughout the United States, was found to be unwieldy, and, at the suggestion of the Committee on Special Committees, the Special Committee on a National Water Law was reduced in number from nine to three members, and thus continued during 1916, a Progress Report being submitted and printed in the *Proceedings* for December, 1916, pages 1937-1942. It was then shown (page 1941):

> "That the Committee has entered upon a course which, for results, demands long-continued, systematic, and intelligently directed efforts, mainly toward the diffusion of information, to be followed by discussion of these large matters by the great body of engineers. Any attempt to initiate legislation, especially that far-reaching in character before the engineers as a whole have fully considered the underlying principles, would be unfortunate."

The Committee was continued in its work during 1917, with a view to developing many of the details in which the matter in hand had ramified.

* To be presented to the Annual Meeting, January 16th, 1918.

Enlargement of Scope.—Following the Annual Meeting of the Society in January, 1917, the "Committee [of the Board of Direction] on Special Committees" stated that in view of its opinion that:

"It is no function of an engineering society to prepare water laws, either State or Federal, and that the whole question of legislation and preparation of laws can better be dealt with by other professional organizations, the present title of this Committee is a misnomer, and the Committee should be rechristened"

and suggested a "Special Committee on the Regulation of Water Rights." This action was taken with a view to "the enlargement of the scope and field of the investigations of this Committee, to cover sources of water, pollution of water supply, irrigation and other uses of water, in addition to its use for power development."

This recommendation having been favorably acted on, the Special Committee revised its field of work accordingly, but still adhering to the consideration of the principles involved in the matter, as indicated in the Progress Report printed in December, 1915, as these principles were fundamental to the details previously mentioned. About this time, the upheaval resulting from the declaration of war with Germany distracted the attention of various co-operating agencies and of individuals, rendering it impracticable to continue the orderly discussion, and indefinitely delayed the completion of matters in hand.

Change of Viewpoint.—It should be emphasized that the past year has been one, not only of titanic preparation for war, but of unprecedented change in our conception of civil matters and relations. The powers and duties of the Federal Government have been greatly magnified, and the statement of principles which a short time ago were considered as almost revolutionary, are now accepted as axiomatic. The ideas concerning the regulation of water rights and the duties of the Federal Government in this matter are involved in this change. To a small extent, at least, the statements enunciated by the Committee on a National Water Law have been effective, inasmuch as they have been accepted and are passing into current use among engineers and others concerned with fundamentals.

Legislative Committee.—The Board of Direction, on January 15th, 1917, as shown by the Minutes* of the Meeting, appointed Messrs. Clemens Herschel, William H. Bixby, and William H. Yates to represent the Society at any conference at Washington on the subject of Water Power Development. Attention at this time is called to the existence of this other committee, in order that there may be no confusion in the matter. The Special Committee on the Regulation of Water Rights is entirely distinct in its functions, and has to do, as above stated, only with the enunciation of the underlying principles,

* *Proceedings*, Am. Soc. C. E., February, 1917, p. 74.

and is not urging action on any congressional bill, its idea being, as stated in those preliminary reports, to bring about a discussion of fundamentals and acceptances of these, rather than advocacy of any particular form of legislation to embody them.

Supreme Court.—Besides the delays which have arisen from war conditions, there has been another important contributing cause, namely, the fact that the Supreme Court of the United States has now before it the so-called Wyoming-Colorado case in which are brought up for judicial decision some applications of the principles which are being discussed by various co-operating societies. Pending the presentation of these and action on them by the Court, it has been deemed wise to defer further consideration of them.

Elaboration of Details.—In several matters, growing out of the principles enunciated in the previous Progress Report, there has been notable advance, particularly in connection with interstate waters. The Committee and some of its co-operating agencies are in practical accord regarding the following statement pertaining to such water, and are offering for further discussion certain propositions substantially to the effect that:

(*a*).—All questions of rights to the use of the waters of any stream or body of water flowing from one State into another or situated on any boundary between States shall fall within the original jurisdiction of the United States District Courts for that area, such waters being referred to generally as "interstate waters."

(*b*).—All persons in any State claiming rights to the use of interstate water for beneficial purposes may be made parties to this suit and, after final determination, all should be governed by this decision.

(*c*).—If rights claimed in two or more districts are involved, the case should be heard before the judges of the corresponding District Courts organized into what may be termed a "Water Right Court", the case to be heard by at least one-half the number of judges constituting such Water Right Court.

(*d*).—Thus, in cases arising along a river which flows along or through several States or Court districts, all the United States District judges within the area involved should sit together in such Water Right Court, and with a view to final determination of these matters, all parties interested should be brought into the suit.

(*e*).—The appeal should be made to the Circuit Court of Appeals if only the circuit is involved, otherwise to the Supreme Court.

(*f*).—It is also recommended that if such Water Right Court be authorized by law, it give due consideration to the infor-

mation affecting the matter in controversy collected by any State or Federal agency, and also receive records of adjudication by any State tribunal concerning water rights involved in a proceeding brought before it.

Although the foregoing is merely a tentative presentation of this important matter, it indicates the character of discussion which is going on concerning water rights and their regulation. It is recognized that progress must necessarily be slow, especially during this period when the minds of engineers and their associates are turned almost exclusively to war conditions. Each step needs to be carefully studied and discussed, and though, on one hand, it is important to continue to keep before the eyes of engineers these important matters, yet, at the same time, it is not desirable to push consideration beyond a certain rate of progress consistent with full deliberation when opportunity arises.

Because of this condition, your Special Committee on the Regulation of Water Rights, following in the footsteps of the previous Special Committee on a National Water Law, respectfully recommends that it be continued.

F. H. NEWELL,
Chairman.

COMMITTEE:

W. C. HOAD,
JOHN H. LEWIS,
F. H. NEWELL.

NOVEMBER 12TH, 1917.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

AIR TANKS ON PIPE LINES

Discussion.*

By L. R. Jorgensen, M. Am. Soc. C. E.

L. R. Jorgensen,† M. Am. Soc. C. E. (by letter).‡—This subject of air tanks on pipe lines is entitled to receive greater attention in the future than it apparently has in the past. There seems to be no reason why such installations should not be of advantage, especially where turbines are the prime movers in the power-house. Undoubtedly, there is a field for air tanks on modern pipe lines, as they act as safety valves in addition to whatever relief device may be provided for the turbines. Mr. Warren's paper is of much value to hydraulic engineers. On high-head plants where impulse wheels are used, the proposition is somewhat different, especially if the needle regulating the size of the jet is hand-operated and the governing is effected by deflecting the jet away from, or into, the wheel buckets. On such a pipe line, there would be less need for air tanks, as the changes in the velocity of the water in the pipe line are under hand control. Mr. Jorgensen.

The writer was once connected with a high-head hydro-electric plant in Southern California where the static head was more than 1 900 ft. There were several air tanks along the pipe line, each tank connected to the line by a short piece of pipe and provided with a valve. Soon after being put into use, these tanks filled up with water completely, as no compressor arrangement had been provided to furnish the necessary air to compensate for air absorption and air leakage from the tanks. As it was also found, however, that the pipe line operated satisfactorily without the air tanks, no compressor was put in. Since then some of the air tanks have been removed and others have been simply

* Discussion of the paper by Minton M. Warren, Assoc. M. Am. Soc. C. E., continued from November, 1917, *Proceedings*.

† San Francisco, Cal.

‡ Received by the Secretary, November 12th, 1917.

Mr. Jorgensen.

disconnected. This plant is regulated by deflecting needle nozzles, the needles being operated by hand whenever great changes of load take place. Had the governor operated the needle direct—in connection with an auxiliary needle for relief—it is very probable that the air tanks would have been advantageous in regulating the pressure in the long pipe line. The erection of a compressor and air pipe system to make the tanks operate properly would not have been much of an undertaking.

A short time ago, the writer had occasion to inspect another high-head plant (1 800 ft.) in California, in which the needle inside the nozzle was operated by the governor direct—in connection with an auxiliary needle for relief. There were strong indications that an air tank on this line would have been very useful in preventing excess pressure, especially at times when the load was low and going still lower.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

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DETENTION RESERVOIRS WITH SPILLWAY OUTLETS AS AN AGENCY IN FLOOD CONTROL

Discussion.*

By Adolph F. Meyer, M. Am. Soc. C. E.

Adolph F. Meyer,† M. Am. Soc. C. E. (by letter).‡—General Chittenden's invitation to all who are interested in flood control to "contribute from their experience or study" of the subject, has prompted the writer to prepare the following notes. Mr. Meyer.

In a recent report by Mr. Arthur V. White and the writer, Consulting Engineers to the International Joint Commission, in connection with the proposed regulation of the level of the Lake of the Woods and the utilization of its waters, and also in the final report of the International Joint Commission to the Governments of the United States and Canada, the combined use of a great reservoir for at least three purposes is specifically set forth. Without going into a detailed consideration of the problems dealt with in these reports, a brief statement may be made of the proposed combined use of the Lake of the Woods reservoir, for several storage purposes. Although the problem was primarily one of storage for public and private uses, the practicability of such use was really contingent on the successful control of extreme flood inflows into the reservoir. The utilization of a natural lake as a reservoir for artificial storage for use almost invariably involves an increase in flood stage on the lake itself, in the channel below the outlet of the lake, or both. Although foresight may prevent these increases in flood stage, there is no assurance that it will, hence provision must be made for the conditions mentioned.

* Discussion of the paper by the late H. M. Chittenden, M. Am. Soc. C. E., continued from November, 1917, *Proceedings*.

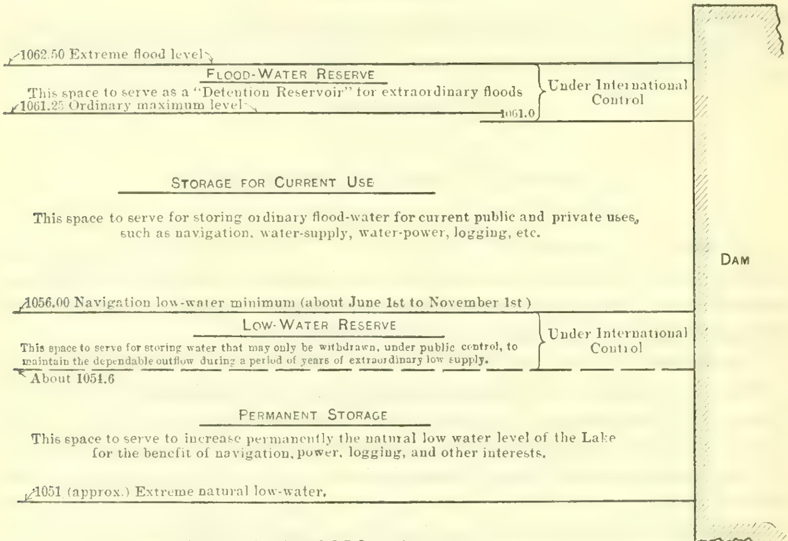
† Minneapolis, Minn.

‡ Received by the Secretary, November 5th, 1917.

Mr. Meyer.

In the present instance, the "top storage", instead of being the cheapest,* is by far the most expensive. This is a condition which commonly prevails on large reservoirs that utilize natural lakes as sites. The present problem, also, is not a "headquarters problem." The water-shed tributary to the Lake of the Woods aggregates 26 750 sq. miles. The mean reservoir surface area is 1 485 sq. miles, and the storage capacity is 41 400 000 000 cu. ft. per ft. depth on the Lake.

The manner in which the available reservoir space was sub-divided in order to secure the greatest aggregate advantage to all interests, both public and private, utilizing the waters of this Lake and the



LAKE OF THE WOODS RESERVOIR
Illustrating Combination of Reservoir Uses.

FIG. 3.

shores and harbors thereof, may be better understood by reference to Fig. 3. The low-water level of the reservoir is to be held at about 5 ft. above natural extreme low water, so as to provide a minimum, during the navigation season, of 1056.0, sea-level datum; this minimum stage will also maintain permanently an increase in the available head at the outlets, reduce the cost of getting water to the power-plants at the outlets, and permit of a more satisfactory operation of saw-mills and power-plants in time of low water. Of the permanent storage below Elevation 1056, a portion is to constitute a "low-water reserve" for use only under international supervision and control, and for the

* *Proceedings*, Am. Soc. C. E., September, 1917, pp. 1437 and 1439.

purpose of maintaining the dependable outflow during a period of years of extraordinary low supply. Mr. Meyer.

The storage capacity between Elevations 1056 and 1061.25, sea-level datum, is to be utilized for storing ordinary flood-water for current public and private uses, such as navigation, water-supply, water-power, logging, etc. The head available for power development at the immediate outlet of the reservoir is only about 20 ft., but an aggregate of about 290 ft. of utilizable fall is partly developed on the Winnipeg River below the outlet of the Lake. Only a small portion of the storage space between Elevations 1056 and 1061.25 will be used in any one year. The intention is to equalize the outflow over a long period of years to a dependable quantity of from 10 000 to 12 000 cu. ft. per sec. The average rate of outflow from the Lake during the past 24 years was substantially 16 300 cu. ft. per sec. The extensive equalization of outflow proposed will be better realized by the fact that, if such regulation had been in operation in the past, the reservoir would not have been full once during the dry period which lasted from the spring of 1910 to the spring of 1916. During this entire period no water would have been wasted. The lowest stage in the reservoir would have been reached in the spring of 1914. Notwithstanding an extreme fluctuation in lake stage of more than 6 ft., the annual storage range for 75% of the time during the past 24 years would have been less than 2 feet.

The upper 1.25 ft. of storage capacity is to be utilized as a "detention reservoir" or "flood-water reserve", under international control. Water stored in this space is to be wasted as soon as possible. It is the expectation, however, that the lower 3 in. of this flood-reserve storage capacity may frequently be utilized for storing water for use on a falling lake stage, after all danger of floods has passed.

Automatic control of flood discharge was not practicable in this instance. The regulating dam, consisting of a rock fill, and piers and sluices with stop-logs, is in a rock channel about a mile below the natural outlet of the lake. With the dam wide open, physical control of outflow virtually passes to constricted portions of the channel above the dam, where extensive rock excavation is required under the proposed control. Under automatic control of flood discharge, reliance is placed on increased water stages, both in the reservoir and at the spillway dam, to increase the discharge. In the present instance, substantially all the available fall in the main outlet channel is used in bringing the flood-water to the dam. In other words, increased discharge is accompanied by a drop in water level at the dam, instead of a rise. The cost of automatic spillway control in this case would be prohibitive. Moreover, the interests on the river below the outlets of the lake make the specification of a definite, maximum, flood discharge from the reservoir highly desirable. The solution recom-

Mr. Meyer. mended is to dispose of ordinary flood inflow into the reservoir by storage, so far as practicable, below Elevation 1061.25; then to increase the discharge until the maximum permissible rate on the river below the dam is reached; and then merely to "detain" the remaining excess inflow in the space reserved for extraordinary floods.

The extreme flood stage of 1062.5 in the reservoir is equal to extreme natural high water in the lake recurring at intervals of perhaps 25 or 50 years. The extreme flood discharge capacity recommended exceeds the natural extreme flood stage on the river below the outlet of the lake by about 1 ft.

The writer trusts that these most essential facts will permit of a reasonably good understanding of the problem of combining several storage projects at one reservoir site on the Lake of the Woods, although it is manifestly impossible to condense into a few pages all relevant material from the before-mentioned four-volume report.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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PAPERS AND DISCUSSIONS

THE SUBSIDENCE OF MUCK AND PEAT SOILS IN SOUTHERN LOUISIANA AND FLORIDA

Discussion.*

BY MESSRS. RUDOLPH HERING AND J. F. COLEMAN.

RUDOLPH HERING,† M. AM. SOC. C. E.—This paper forms an excellent contribution on a subject which has not as yet been fully covered, and, having had some experience with it in the City of New Orleans, the speaker desires to add a few remarks thereon. On both sides of the city lie some of the shaded areas referred to by the author. About 27 years ago, it was decided to build a sewerage system in New Orleans; but there was much opposition to the usual method of design. It was feared that the city buildings would collapse by the drying and settling of the soil in consequence of under-drainage. Mr. Hering.

A system was suggested by the late Col. George E. Waring, Jr., of keeping the sewers quite near the surface and pumping the sewage frequently. Another was suggested by Mr. Broughton, adopting the Shone system, which, at frequent intervals, caused the sewage to be pumped automatically by compressed air. A profile of a sewer built on such a system would look like a saw.

The speaker was engaged in 1892 to give an opinion on these propositions, somewhat novel and covering a difficult problem. It was suggested to make some excavation tests. The ground-water level in New Orleans stood quite near the surface, and the material consisted of clay and very fine sand, with some organic matter. The clay was more than half of the bulk. Consequently, it was impracticable to have any cellars in the city, and, in the cemeteries, the bodies were buried in mounds above the general surface.

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* Discussion of the paper by Charles W. Okey, Assoc. M. Am. Soc. C. E., continued from November, 1917, *Proceedings*.

† New York City.

Mr Hering. To drain this soil would mean shrinkage, and the question was, how much and how soon? A test pit was dug, about 5 ft. square and about 25 ft. deep. It was found that one man excavating could also keep the pit bailed out with a bucket. The water oozed out of the material very slowly; at no place was there any current. In a very few days, the pit, if left to itself, would fill with water.

It was concluded that if the sewers were built rapidly, so that the trenches would not remain open long, they could be readily built, even at a depth of 25 ft. The whole system was designed accordingly, built with no special trouble, and is in a satisfactory condition to-day. Along one block on Canal Street, the trench was open for some time, and a crack occurred parallel to the street several hundred feet away.

George G. Earl, M. Am. Soc. C. E., the Chief Engineer, kept very close watch of the work and the amount of damage on such an extensive and difficult job was very small. It was naturally deemed essential to have the sewers as tight as practicable, not only because of the natural drainage, but also because of the necessary pumping of all the sewage.

There were many heavy stone buildings in the city, but in most cases they were founded on piles or grillage. In special cases, it was believed that it might be required to deepen the foundations or protect them from deterioration. It has proved that, up to the present, the gradual and slow draining of the subsoil has enabled the necessary precautions against collapse to be taken, and very little serious trouble has been caused. It has also been proved that it is now feasible to have cellars for the buildings.

The speaker believes that these good results are due partly to the good construction of the sewers, partly to the fact that the surface is mostly impervious—owing to the buildings and good pavements and the quick removal of rain water by a special drainage system—and partly to the watchfulness of the authorities.

To build a sewer system in a material that was practically "muck" was certainly an interesting proposition. Of great importance as preliminaries, in the speaker's opinion, were a physical analysis of the soil, a minimum length of open trench, and rapid but very careful construction.

The speaker admits what Mr. Coleman says, relative to the difference in the soils described, but the author's remarks apparently refer also to the Areas 35, 36, and 37 indicated on the map, and these immediately adjoin the City of New Orleans. The soils, therefore, are probably not very different. The speaker wishes to add that, as it is impracticable with ordinary sewer construction to have the sewers absolutely water-tight, the ground under the city would eventually and gradually drain itself. It was concluded that the shrinkage, so

far as it would go, would have to be very slow, and could and should be observed and suitable provisions made to prevent any serious disturbances. Mr. Hering.

J. F. COLEMAN,* M. AM. SOC. C. E.—The speaker has been quite familiar with the author's investigations throughout the entire period of the operations in these peat soils, and has been kept informed of all the work which he has done, and with what the governmental department expects to do. The speaker fully agrees with the remarks of Mr. Morgan to the effect that the information to be gathered by this series of investigations and measurements will be of inestimable value to those who have to deal with drainage and reclamation projects on these peat and muck lands. Mr. Coleman.

The speaker has listened with considerable interest to the remarks of Mr. Hering on the subject; but feels quite sure that Mr. Okey's paper does not intend to deal with the particular kind of soil which Mr. Hering and his associates encountered in connection with the sewerage and drainage of the City of New Orleans.

The judgment which they expressed to the City authorities has been amply sustained since that time for the reason that there has been no appreciable subsidence of buildings in New Orleans, due to the subsidence of the soil on which they rested; and the only troubles from which buildings have suffered as a result of this drainage has been due to the fact that many of the old ones, which were constructed on spread foundations, had those foundations imposed on grillage of timber; and, in some instances—the number of instances is rapidly multiplying now—the lowering of the line of saturation has caused a decay in the timber grillage, and some foundations are beginning to need treatment as a result.

The soil, however, with which Mr. Okey's paper deals, may be more fairly denominated as peat, in so far as Southern Louisiana is concerned. It is a soil which, under no circumstances, would carry a greater load for spread foundations, for example, than from 100 to 250 lb. per sq. ft., whereas the soil of New Orleans, in the area that has been dealt with in the drainage and sewerage system, will support from 750 to 1 500 lb. per sq. ft.

The soil of New Orleans has been overflowed, in years gone by, by the Mississippi River, and portions of the original muck soil there have been very completely filled over with sand and silt, which would be indicated by the analysis which Mr. Hering mentioned, showing about 50% of sand and quite a considerable quantity of clay in addition; whereas, in the peat or muck soil of Louisiana, as described in Mr. Okey's paper, there is virtually no sand.

* New Orleans, La.

Mr. Coleman.

On such land, after the water-table has been reduced by the pumping station, and has been thus maintained for a period of months, it takes little or nothing to set fire to that soil during a dry spell; and it will burn down to the water-table. In fact, to fight these fires they excavate ditches surrounding the fire down to the line of saturation; so that they are truly, or almost truly, peat lands. The material may be excavated from them and made into briquettes for fuel.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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PAPERS AND DISCUSSIONS

PULSATIONS IN PIPE LINES, AS SHOWN BY SOME RECENT TESTS

Discussion.*

By Norman R. Gibson, Rudolph Hering, and R. D. Johnson.

Norman R. Gibson,† M. Am. Soc. C. E. (by letter).‡—This paper is one of the most interesting and instructive that have come to the notice of the writer, who, during the past year, has had occasion to make a careful study of the subject. The results of the experiments are evidently reliable, and they supply the much needed data with which analytical calculations may be verified. With many of the conclusions of the author the writer is in accord, but not with regard to the general application of his formula proposed for pressure variation from normal. In the writer's opinion, Mr. Vensano's error lies chiefly in the second assumption in the note on the curve shown in Fig. 28, namely, "penstock velocity assumed to vary directly as the time of opening or closing nozzle"; and this assumption, apparently, has led to an incorrect interpretation of the data obtained from his experiments. The close agreement between the calculated values of p and the values obtained by the experiments is due to the fact that the head acting on the nozzle was very high, in which case the retardation of the flow of water in the pipe was more nearly uniform than it would have been under a lower head. To apply the author's formula to pipes under comparatively low head, however, would lead to results which it is doubtful would be confirmed by experiment. Mr. Gibson.

The influence of the net head on the flow of water in pipes and the consequent rise of pressure caused by gradually stopping the flow,

* This discussion (of the paper by H. C. Vensano, M. Am. Soc. C. E., published in October, 1917, *Proceedings*, and presented at the meeting of November 7th, 1917), is printed in *Proceedings*, in order that the views expressed may be brought before all members for further discussion.

† Niagara Falls, Ont., Canada.

‡ Received by the Secretary, November 7th, 1917.

Mr. Gibson.

has been made the subject of a paper prepared recently by the writer, in which formulas are derived from Joukovsky's formula, $h = \frac{V\,a}{g}$, for instantaneous water-hammer. This paper at the present time is undergoing revision, by request, to make it more easily understood, and it is hoped will soon be in shape for publication. The treatment of the subject as contained therein is too long to be introduced here as a discussion of Mr. Vensano's paper, but the results of his experiments are so closely in accord with the writer's analytical results that it is hoped a brief explanation of them will be found helpful in studying the data.

The influence of the net head on the phenomena that occur in a pipe when the flow is being arrested cannot be neglected, because the rate of retardation of the flow depends on two factors: first, the closing of the gate, and second, the head acting on the orifice at the gate. In other words, the velocity of the water in the pipe depends on the area of the gate opening and the net head acting on that opening. The magnitude of the rise of pressure caused by gradually closing the gate depends on the velocity destroyed, the rate at which it is destroyed, the length of the pipe line, the compressibility of water, and the elasticity of the pipe. The last two factors are taken into consideration in the velocity of the pressure wave, *a,* as determined by Joukovsky, with which all students of this subject are familiar. For an instantaneous closing of the gate, the magnitude of the pressure rise depends only on the velocity destroyed, the compressibility of water, and the elasticity of the pipe.

The relation between the velocity in the penstock, the area of the gate opening, and the net head, may be expressed by the equation,

$$V = B\sqrt{H} \qquad (1)$$

where V = velocity of flow in pipe of uniform diameter, in feet per second;

H = net head acting on the orifice, in feet;

B = simply a number representing the gate opening.

During the closing of the gate, all three of these quantities vary. V and B become zero, and H rises to its maximum value. The rate at which B varies is known from the movement of the gate. The rate at which V varies is unknown, and depends on the increase in the value of H, which is also unknown. At all times during the gate opening, however, the relation of the three variables, as expressed by the equation, must be maintained.

Joukovsky's formula for the rise of pressure caused by an instantaneous closing of the gate in a pipe is

$$h = \frac{V\,a}{g},$$

Mr. Gibson.

where h = rise of pressure, in feet;

a = velocity of the pressure wave, in feet per second;

g = acceleration due to gravity.

For a slight instantaneous change in velocity,

$$\Delta h = \frac{a}{g} \Delta V \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2)$$

The determination of the value of a has been explained by the author, and need not be repeated here. As already stated, the derivation of the writer's formulas for the rise of pressure caused by the gradual closing of turbine gates, is too long to be given here, and the formulas themselves are too complex to be used without a lengthy explanation. Using Equations (1) and (2), however, a numerical problem may be solved by the trial and error method of arithmetic integration for the conditions under which Mr. Vensano's experiments were performed. The calculated results may then be compared with those obtained by experiments.

It is unfortunate that the pipe line described by the author is of varying diameter, as this complicates the problem at the outset, and no doubt accounts to some extent for the difference he found between the calculated and observed values of the wave period. The observed value of the wave period, namely, 6.95 sec., will be used in the following calculations, and from it is found the value of a, as $\frac{4\ L}{a} = 6.95$ sec.

Therefore, $a = \frac{4 \times 6337.42}{6.95} = 3\ 647$ ft. per sec.

After determining the value of a by calculation or experiment, it would appear reasonable to assume that the pipe was of uniform diameter, such that the velocity in it was a weighted mean of all the velocities in the various sections with respect to their length. That is to say, the sum of the products, $L_1\ V_1$, $L_2\ V_2$, $L_3\ V_3$, etc., divided by the total length of the pipe, would give a mean velocity, which, if gradually destroyed, would cause the same pressure rise as would be caused by the destruction of the various velocities in the pipe as it actually existed. Such an assumption may be questioned, but it simplifies the labor of making the calculations, and in this case, at least, seems to be justified by the results.

Table 3 presents the pipe data, from which the mean velocities were thus found to be:

15.055 ft. per sec. when Q = 365 cu. ft. per sec.
7.672 " " " " Q = 186 " " " "
3.877 " " " " Q = 94 " " " "

The calculations to determine the rise of pressure for the conditions under which the author's Experiments Nos. 15 and 16 were performed, are tabulated in Table 4. These may be explained as follows:

Mr. Gibson. The static head with gate closed was 582 lb. = 1 341 ft. The gauge reading at the gate was 540 lb. = 1 244 ft. To this must be added the velocity head, due to the velocity at the gate, which was 32.4 ft. per sec. The velocity head, therefore, was 16.3 ft., and the total net head producing discharge through the orifice was 1 260.3 ft. The head lost in friction under the full flow was 1 341 — 1 260 = 81 ft. The value of B may now be found from Equation (1),

$$B = \frac{V}{\sqrt{H}} = \frac{15.055}{\sqrt{1\,260}} = 0.4241.$$

TABLE 3.—PIPE DATA Q = 365.

Division.	Diameter, in inches.	Area, in square feet.	Velocity. V	Length. L	LV
1...............	20–26	2.93	33.30	4.0	133.2
2...............	26	3.687	24.740	33.25	825.0
3...............	36	7.068	25.818	18.0	464.7
4...............	52	14.748	24.740	194.17	4803.7
5...............	54	15.904	22.950	455.4	10451.5
6...............	60	19.635	18 589	728.0	13532.9
7...............	66	23.758	15.363	768.0	11798.8
8....	72	28.274	12.909	4136.6	53400.2
				6337.42	95410.0

$$\text{For } Q = 365 \text{ Mean } V = \frac{95\,410}{6337.42} = 15.055 \text{ ft. per sec.}$$

For Q = 186 Mean V = = 7.672 " " "
For Q = 94 Mean V = = 3.877 " " "

The rate at which B varies is not uniform, as is shown by the author's curve on Fig. 6. From this curve the time of gate closure is shown to have been about 69 sec. During that time the pressure wave started by the first movement of the gate would travel up to the forebay and back at the rate of 3 647 ft. per sec., and, therefore, would make 19.88 return trips. For the sake of simplicity, assume that 20 return trips were made. It should be remembered that the velocity of the pressure wave is constant and independent of the magnitude of the wave. Now assume that the gate was closed in 20 instantaneous movements, each movement occurring regularly every $\frac{2\,L}{a}$ seconds. The time, $\frac{2\,L}{a}$, will be called one "interval."

The value of B at each movement of the gate may now be determined from the author's curve in Fig. 6, by dividing the time base into 20 even spaces and multiplying 0.4241 by the ratio of the area of nozzle opening opposite each division to the full area of nozzle opening. The result should be plotted on a curve, and, after smoothing out

the irregularities, the values of B may be taken off and recorded, as shown in Column 2 of Table 4. Mr. Gibson.

TABLE 4.

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
Interval.	Gate. B	Head. H	Velocity. V	$113 \Delta V$ Δh	$\Sigma \Delta h$	$0.3574\ V^2$ h_f	Δh_f	$\Sigma (\Delta h + \Delta h_f)$
0	0.4241	1260.0	15.055			81.0		
			0.155	17.5				
1	0.4168	1279.2	14.90		17.5	79.3	1.7	19.2
			0.29	32.8				
2	0.4084	1279.8	14.61		15.3	76.5	4.5	19.8
			0.28	31.7				
3	0.3995	1284.0	14.33		16.4	73.4	7.6	24.0
			0.33	37.3				
4	0.3895	1291.8	14.00		20.9	70.1	10.9	31.8
			0.37	41.8				
5	0.3784	1295.5	13.63		20.9	66.4	14.6	35.5
			0.42	47.5				
	0.3657	1305.3	13.21		26.6	62.8	18.7	45.3
			0.49	55.4				
7	0.3518	1311.8	12.72		28.6	58.0	23.0	51.8
			0.53	59.8				
8	0.3354	1319.0	12.19		31.0	53.0	28.0	59.0
			0.60	67.8				
9	0.3180	1329.8	11.59		36.8	48.0	33.0	69.8
			0.65	73.5				
10	0.2991	1334.9	10.94		36.7	42.8	38.2	74.9
			0.72	81.3				
11	0.2781	1348.2	10.22		44.6	37.4	43.6	88.2
			0.84	94.8				
12	0.2544	1359.7	9.38		50.2	31.5	49.5	99.7
			0.92	104.0				
13	0.2285	1369.2	8.46		53.8	25.6	55.4	109.2
			0.97	109.5				
14	0.2009	1376.6	7.49		55.7	20.1	60.9	116.6
			1.07	121.0				
15	0.1719	1391.6	6.42		65.3	14.7	66.3	131.6
			1.15	130.0				
16	0.1411	1395.8	5.27		64.7	9.9	71.1	135.8
			1 21	136.8				
17	0.1083	1407.2	4.06		72.1	5.9	75.1	147.2
			1.33	150.1				
18	0.0725	1416.3	2.73		78.0	2.7	78.3	156.3
			1.365	154.2				
19	0.0363	1416.5	1.365		76.2	0.7	80.3	156.5
			1 365	154.2				
20	0.	1419.0	0.		78.0	0.	81.0	159.0

Table 4 may now be completed as follows: In the first line opposite $B = 0.4241$, set down in Columns 3, 4, and 7, the known values of H, V, and h_f (friction head). Next assume a trial reduction in velocity, caused by the initial instantaneous movement of the gate, and set the figure down in Column 4 under the value of V, and subtract it from V, placing the difference immediately underneath. This trial figure is ΔV, and is assumed to be destroyed instantaneously by the first movement of the gate. A pressure wave, Δh, of magnitude $= \frac{(\Delta V)\, a}{g} = (\Delta V) \frac{3\ 647}{32.2} = 113\ \Delta V$, is therefore started up the pipe. The

Mr. Gibson.

product of 113 ΔV is set down in Column 5, opposite ΔV. In Column 6 is recorded the algebraic sum of the values of Δh. In Column 7 the total friction head, due to the velocity shown in Column 4, is set down, and in Column 8, the friction head, recovered at each operation, is shown. For convenience, h_f may be made equal to $C\ V^2$, in which

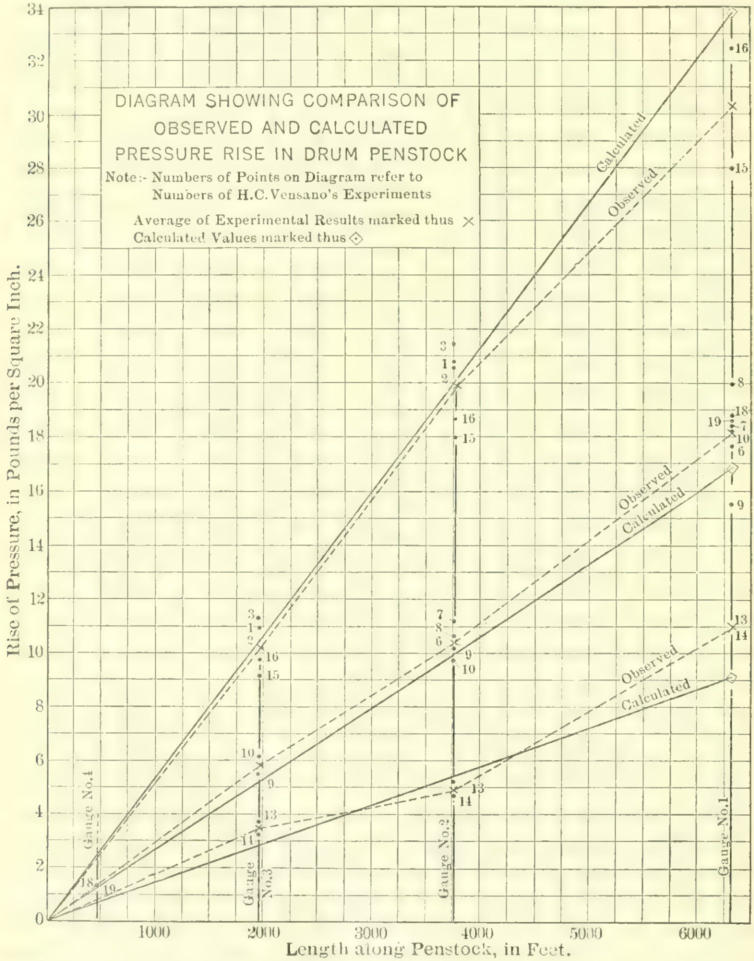

FIG. 32.

C is a coefficient obtained from the known values at the beginning. In the example, $C = 0.3574$. Column 9 shows the sum of the opposite items in Columns 6 and 8. Having obtained the figure in Column 9, it is added to the net head, $H = 1\,260$, and the sum is set down in the

next lower line in Column 3. The result must now be checked to see that $B\sqrt{H} = V$, where B is now 0.4168, and H and V are the values opposite. If the relation is not satisfied, a new trial value of ΔV must be chosen, and the operations repeated until a check is obtained. After trial, the initial value of ΔV was found to be 0.155. The operations for obtaining the figures on the third line are not quite the same as those just described, because, after assuming the next trial value of ΔV, and multiplying it by 113, it must be remembered that the resulting pressure at the gate is reduced by the return of the first wave which has traveled up to the forebay and back, and now changes to sub-normal and repeats the journey. The time at which the gate is given its second movement, has been selected purposely to coincide with the return of the first wave. The item opposite Interval 2, in Column 6, therefore, is the difference between the first two figures in Column 5. The other operations are similar to those already described, and the resulting figures in Columns 3 and 4 are checked similarly with the value of $B = 0.4084$. The succeeding lines are filled in by the same process, always keeping in mind the return of the preceding waves, and whether they change to sub-normal or super-normal. A little study will disclose the fact that the figure in the second line of Column 6, may be obtained by subtracting the figure in the first line of Column 6 from the figure in the second line of Column 5. Similarly, the figure in the third line of Column 6 may be obtained by subtracting the figure in the second line of Column 6 from the figure in the third line of Column 5. Mr. Gibson.

Proceeding in this manner (a task easier to do than to explain, and one which becomes easy with a little practice), the whole of Table 4 is filled out, and the resulting maximum value of H is found to be 1 419 ft. As the hydraulic gradient has arisen to 1 341 ft. during the gate closure, the rise of pressure above normal is 78 ft. = 33.85 lb. The average obtained by Experiments 15 and 16 was 30.25 lb., the maximum being 32.5 lb., and the minimum 28 lb.

The maximum pressures at intermediate points on the pipe line may now be found by plotting Fig. 32, in which the length of pipe line is the abscissa and the pressure rise at the gate is the ordinate. A straight line from the ordinate at the gate to zero at the beginning of the pipe determines the ordinates of pressure rise at intermediate points. At Gauge No. 2, the pressure rise shown by this diagram is 20.2 lb., and the average of the experimental values is 19.9 lb., the maximum being 21.5 lb., and the minimum 18.0 lb. At Gauge No. 3, the diagram shows 10.5 lb., and the average of the experimental values is 10.2 lb., the maximum being 11.2 lb., and the minimum 9.7 lb. On the diagram the other calculated and experimental results have been plotted, from which the close agreement between the two may be noted.

Mr. Gibson.

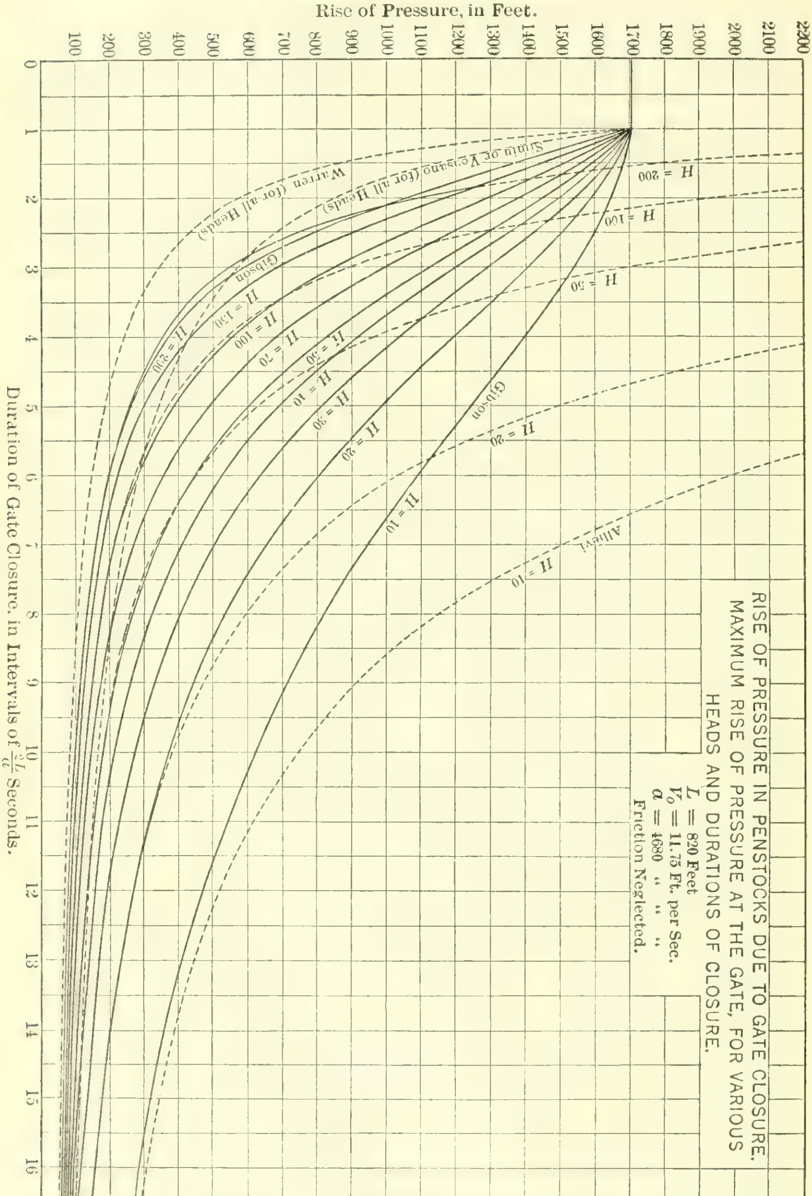

FIG. 33.

The foregoing method may be followed to determine the rise of pressure in a pipe line caused by a shut-down under any conditions. If smaller changes of gate movement are selected, say 40 or 80 movements during the closure, the increments of pressure at each operation become smaller, but it is a singular fact, though easily understood, that the magnitude of pressure rise at the end of each whole "interval" is exactly the same as when the integration is performed by using gate movements one "interval" apart. The formulas developed by the writer are based on the foregoing method, but determine the rise of pressure at the end of each interval, or at any time during the interval, without recourse to trial and error work. It will be noted that this method gives maximum water-hammer for any duration of closure from zero up to 1 "interval," $\left(\frac{2\ L}{a}\right)$. As the duration of closure becomes longer, the resulting rise of pressure obtained by the method finally approaches the value given by Allievi's well-known formula, the curves having the general characteristics shown in Fig. 33. On this figure the corresponding curves obtained from the formulas of Allievi, Vensano, and Warren are shown for comparison. Mr. Gibson.

The writer's conclusion, therefore, is that Mr. Vensano's formula, $h = \frac{2\ L\ V}{g\ t}$ is exactly correct when H is infinite, and is approximately correct when H is very large. The formula is not applicable, however, when H is relatively small.

There are many other interesting features of this subject, which, owing to lack of time, will have to be reserved for future discussion. Among these are the wave forms shown by the graphic charts, a close examination of which seems to show that they possess characteristics similar to the theoretical forms of the waves at the various points along the pipe, the tops of the waves, it will be noticed, becoming flatter as the upper end of the pipe is approached, and sharper as they get nearer the gate.

In conclusion, the writer would like to express his appreciation of the valuable information which Mr. Vensano has presented.

RUDOLPH HERING,* M. AM. SOC. C. E.—This paper deserves favorable comment. It covers an important practical question on which a great deal more information than we now have is needed. Mr. Hering.

The speaker thinks that there is still another aspect of this subject, which is not practically important in the question discussed, but is of interest and perhaps of value in other directions.

A mathematician who delves into this subject from a more general point of view may be assisted in considering the results which are inevitable when two bodies move over each other so that they cause

* New York City.

Mr. Hering. frictional resistance. The subject was referred to by the speaker in the discussion of a paper before this Society many years ago. When air, liquids, or solids move or slide over each other, pulsations are caused, which sometimes are quite strong. It may be remembered that it sometimes takes two firemen to hold the nozzle steady when a fire stream is issuing at a great velocity. Left alone on the ground, the nozzle would vibrate considerably. At the long outfall sewer in Birmingham, England, with no gate, there was a regular oscillation of the water surface near the outlet to the extent of several inches. No explanation was given. A flag flying in the wind shows oscillations caused by the flow of air against a solid. A trailing rope behind a moving boat shows oscillations caused by water flowing against a solid. Unless a ball fired from a gun is given a rotary motion in the barrel, it will not travel a steady course, but will oscillate. True solids moving over each other under favorable conditions cause vibration, generally sufficiently rapid to make a sound. Water dropping through air or forced through a fixed body of water, will not travel like a rod or sheet, but, by friction, will break up into globules which are started by the same forces as cause vibrations under other conditions.

The speaker believes that this is not only an interesting subject, but that its mathematical generalization may help engineers to find better solutions for many practical problems.

Mr. Johnson. R. D. JOHNSON,* ESQ.—There is one point in Mr. Gibson's discussion which seems to deserve emphasis.

Mr. Gibson calls attention to the fact that the actual state of pressure existing in the water column prior to interruption of flow has a marked influence on the rise of pressure due to such interruption.

This statement, of course, is not a novel one, and may seem quite obvious, but neither Mr. Vensano nor Mr. Warren seems to have a proper appreciation of the fact.

The speaker recalls his criticism of Mr. Warren's paper† on this same subject when he called attention to the error introduced by reason of neglect of the prior head, in computing the rise of pressure.

Mr. Warren replied, in summing up, that any formula which varied its results with the static head, must be inherently wrong.

Such diversity of opinion on a perfectly definite matter should not continue to befog these studies, and it is a satisfaction to note that Mr. Gibson has apparently cleared up this point beyond further question.

Any formula, such Mr. Vensano's or Mr. Warren's, which does not involve the prior static head must be inherently wrong, from a practical point of view.

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* New York City.

† *Transactions,* Am. Soc. C. E., Vol. LXXIX, pp. 277–281.

The speaker takes no exception to Mr. Vensano's formula, if and when there is actually uniform diminution of the quantity of water discharged through the gate, during closure; but this is never the case for finite head, with a uniform closure of the gate, and neither is it so for the particular variable rate obtaining in the experimental work which Mr. Vensano considers to justify his formula. Mr. Johnson.

As a theoretical expression of the true rise in pressure in a practical case of slowly interrupted flow, this formula can have no proper place in engineering literature, and should be finally and conclusively discarded.

The method of predetermining pressure rise, as illustrated by the speaker in criticism of Mr. Warren, proven many times experimentally, is practically accurate for slow-closing gates, at ordinary heads, when friction is taken into account, and this factor may easily be considered as there pointed out, although the numerical illustration did not include it.

For rapid-closing gates the well-known formula of Joukovsky must be borne in mind, and just what constitutes slow-closing and what takes place for intermediate rates of closing is apparently demonstrable by such methods as those outlined by Mr. Gibson in his discussion.

If the static head affecting Mr. Vensano's experiments, or the manner of gate motion during closure, had, either or both, been materially different, the rise of pressure would probably not so nearly have agreed with the formula, $2 \frac{L V}{g T}$, in fact, it could not, except by a coincidence of conditions, and this agreement which he finds must be regarded as purely accidental.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

A BRIEF REVIEW OF TRIGONOMETRICAL MATHEMATICAL TABLES, AND A CONTEMPLATION OF THE SPECIFICATIONS FOR TRIGONOMETRICAL TABLES FOR GENERAL USE*

Discussion.*

BY WILLIAM G. RAYMOND, M. AM. SOC. C. E.

WM. G. RAYMOND,† M. AM. SOC. C. E. (by letter).‡—Without doubt, the book proposed by Mr. Eberly would be of value to a certain class of computers, but it is thought that it would be altogether too large for general use. It is proposed to have one degree on a page, with the degree divided decimally. This would require a hundred lines, besides the headings, and would mean a page running from about 14 to 17 or 18 in. long. The book, therefore, would be very unwieldy, and, of course, could not be used at all for field operations. Mr. Raymond.

The writer is a champion of the decimal division of the degree, being the author of the "Field Manual" referred to by Mr. Eberly, and is also a champion of five-place tables for field use, but realizes that seven-place tables are absolutely necessary for certain office computations, although not for the office solution of field problems. Nor is it practicable to use a computing machine for field calculations. Therefore, there must be two classes of tables. Indeed, perhaps, there should be three classes: one set for the great bulk of engineering computations that require not more than four-place tables; these should be of natural functions for use where computing machines are

* This discussion (of the paper by Virgil A. Eberly, Assoc. M. Am. Soc. C. E., published in October, 1917, *Proceedings*, but not presented at any meeting), is printed in *Proceedings*, in order that the views expressed may be brought before all members for further discussion.

† Iowa City, Iowa.

‡ Received by the Secretary, November 8th, 1917.

Mr. Raymond. available, but of logarithmic functions where computing machines are not available; and until computing machines are very much reduced in cost, they will not be generally available for school classes or for offices having only a moderate quantity of computing work. The second class should be five-place tables for field use, and, for this purpose, as computing machines are not available, the tables should be logarithmic. It has been shown that the labor of computation is increased about 50% by the use of five-place instead of four-place tables, and about one-third by the use of six-place instead of five-place tables; and no ordinary field work is done with such precision as to warrant results certain to more than four significant figures, for which five-place tables give sufficiently exact results. Certain city, bridge, and geodetic surveys are made with greater precision, and require more extensive tables. The seeming precision of six-place over five-place tables is, for practically all ordinary field purposes, merely a seeming, but it appears to require great strength of mind on the part of engineers to realize this sufficiently to conform their practice to this truth. In the writer's judgment, there is really little excuse for six-place tables; five-place tables being all sufficient for ordinary field operations and seven-place tables being required for work of higher precision comparatively little work of an intermediate degree of precision being done. Therefore, the third class of tables should be seven-place.

Objection has been made to the use of the decimal division of the degree on the ground that it will cause the same confusion that is imagined by the objectors to the introduction of the metric system; but this is not true, because there is no change of the unit. The unit is the degree, or one-ninetieth of a quadrant, and as American surveyors and engineers have long since adopted for practically all field work the decimal division of the linear unit, there would seem to be no logical reason for not adopting the decimal division of the angle unit. The adoption of the decimal division of the degree would not lessen the value, or interfere with the ready reading, of any old records, but would materially lessen field work and the chances of error in passing from decimal to sexagesimal fractions, an operation that occurs in almost every field calculation involving angles.

Summarizing, the writer supports Mr. Eberly in his first and second propositions; namely, that there should be 90° to the quadrant, and that the degrees should be divided decimally; with his fourth proposition, where tables are to be used for office computation of high precision work; with his fifth proposition that the tables should be of natural functions only in case computing machines are to be used; and, in part, with his sixth proposition, that the book should be thumb-indexed and page numbers omitted; but it is suggested that each page should contain but one-half degree when the arguments are

to one one-hundredth of a degree, in spite of the fact that this would double the number of pages for the angle tables. Proposition VII the writer does not feel competent to discuss. The canons included should cover the needs of all computers of high precision work, but possibly the book should be arranged so that it could be published in parts, not all computers having need of all the functions indicated. Perhaps it seems a bit curious, but the larger the page the larger should be the type, that is, the greater the number of arguments on a page the larger should be the type, so that placing 100 arguments on a page instead of the usual 60 (or 50 for $\frac{1}{2}°$ with decimal division) will increase the size of the page more than in proportion to the number of arguments. Mr. Raymond.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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PAPERS AND DISCUSSIONS

THE HELL GATE ARCH BRIDGE AND APPROACHES OF THE NEW YORK CONNECTING RAILROAD OVER THE EAST RIVER IN NEW YORK CITY

Discussion.*

By Messrs. W. H. Breithaupt, Leon S. Moisseiff, and Samuel T. Wagner.

Mr. Breithaupt.

W. H. Breithaupt,† M. Am. Soc. C. E. (by letter).‡—This magnificent structure, well described as of imposing magnitude, adds but another, if not the greatest, to the long list of achievements of its noted designer and chief engineer. The bridge, as likewise the whole New York Connecting Railroad of which it forms a part, will rank among the great things done in American engineering.

Mr. Ammann's paper is a valuable addition to the literature of bridge engineering in its record and analysis of the bold design as a whole, and the clear-cut account of details and new departures, among which may be mentioned the 2-in. web plates and make-up of the main compression members, the large rivet sizes, the main arch bottom chord joint detail, the braking resistance, etc.

In the face of such excellence of design and execution, it appears graceless to criticize; however, a few remarks on the general design may be in place.

The dignified and beautiful abutment towers of the main arch are wholly admirable, more especially when compared with the earlier design, Fig. 8, with its suspended sign-board keystone, its meaningless surface elaborations, its flaring base, its lack of form as a whole.

* This discussion (of the paper by O. H. Ammann, M. Am. Soc. C. E., published in October, 1917, *Proceedings*, and presented at the meeting of November 21st, 1917), is printed in *Proceedings*, in order that the views expressed may be brought before all members for further discussion.

† Kitchener, Ont., Canada.

‡ Received by the Secretary, November 21st, 1917.

Mr. Breithaupt.

For the Little Hell Gate Crossing, deck-truss, parallel-chord spans, as at first designed, but ruled out by the War Department on account of navigation requirements, would have made the most fitting structure. Arches are not discussed. It is not clear why a two-hinged, spandrel-braced arch design, or two-hinged, braced, double-rib arches, should not have been used. A low-spring, high versed-sine arch design would have effected a considerable saving in masonry with little or no addition in superstructure weight. Aside from the floor support struts, high at the ends, the superstructure frame could have been of less weight. Erection could have been balanced outward from the piers—tied from end abutments—the material floated to position underneath and hoisted, with only a few bents under end-panel points each way, thus effecting a large saving in expensive falsework. The skew interference could have been taken care of as well as, or better than, with the bowstring trusses. The bracing could have been better. The bowstring spans have no lateral bracing along the bottom chords; the stiff bracing between the trusses, although without bottom transverse struts, is, in the description of the main arch bracing, rightly deprecated as tending to distortion under one-sided loading. Expansion would have been taken care of separately for each span. The relative movement of ± 6 in. at the center pier of the bowstring spans, as described, is unduly large. With arches, the high clearance for navigation would have been at the center of the span, with plenty of room for lateral movement of tow, instead of danger of crowding against the piers, as with the present spans. On the other hand, the present spans, for the greater part of their length, give fair clearance for ordinary navigation. For navigation, in fact, as well as for general appearance, three arches for the crossing would have been the preferable design. Lastly, there is the reason, the more deserving of consideration in so monumental and visually prominent a structure, of harmony of general design. The inverted arch effect of the bowstring girders, so near the main arch, involves something in the nature of mental gymnastics.

The gate-post design of the towers marking the ends of the Little Hell Gate Crossing is not entirely satisfactory, either in photograph or to the eye *en plein*. A conceivable purpose would have been as shelters and material or implement stores for watchmen, sentries, or workmen, and a design, lower while retaining massiveness, would have been in keeping.

Mr. Moisseiff.

Leon S. Moisseiff,* M. Am. Soc. C. E.—This paper is of unusual importance in the literature of bridge building, and deserves attention and study. It occurred, under a fortuitous constellation, that a first-class railroad, provided with ample financial resources, undertook to build a short connecting line in the metropolis of the Western hemi-

* New York City.

Mr. Moisseiff

sphere. It, of course, decided to build the new line in a first-class manner; and, as part of the line happened to require the erection of a long-span bridge capable of carrying a very heavy traffic, the management of the road, in its wisdom, selected a first-class bridge engineer with a broad vision and decided scientific leanings. Adding the opportunity of ample time to conceive, to plan, to compare, to design, and to re-design, and again adding to the plans and specifications the resources in mind and matter of one of the greatest bridge manufacturing concerns of the world, the resulting product is such that American engineers may be proud of the achievement. The longest arch bridge in the world, having a span of nearly a thousand feet, has been planned and built so as to produce an imposing and pleasing structure, well proportioned in outlines as well as in details.

The paper is full in its description and, as it takes up successively the consideration and treatment given to the design in general and to its details, numerous points are brought out worthy of commendation and discussion. Many of the vital points of bridge engineering are touched, too many indeed to be given the discussion they deserve.

The comparative designs of the several types of bridges, all with a central span of 850 ft., are most instructive. The stiffened suspension type with eye-bar chains has shown off remarkably well, for a span comparatively small for a suspension bridge, such bridges being best adapted for the longest spans. The good showing made by this type is due to the eye-bar design. If built, it would have resulted in a fine and economic bridge. Somehow, from reading the paper, one obtains the impression that if the difficulties encountered in one of the abutment foundations had been known at the time of selecting the type, the suspension bridge with a longer span, and not the arch, would have been chosen.

The difficulties encountered in the foundation of the Wards Island abutment were unusual, and, if known, the foundations would certainly have been avoided. The appearance of the masonry towers flanking the steel arch is good, but their use, to restrict the size of the foundation and by their weight to obtain a steep resultant to pass within the middle third of the base area, can hardly be claimed to be economical.

Very properly the bridge was erected as a three-hinged arch for most of the dead load and was made two-hinged for moving load and temperature only. In connection with this the author remarks: "This, however, is a convenience in erection rather than advantage as regards stress action." This statement is true where the abutments are absolutely unyielding, but, where any limited settlement may be apprehended, erecting the arch first as three-hinged has a more substantial advantage than mere convenience of erection. It has the purpose of eliminating stresses caused by yielding, which stresses may become uncomfortably large. In that case it is good policy to leave

Mr. Moisseiff.

the arch three-hinged for at least a year, so that all probable settlement and yielding may have taken place and be eliminated, and, after that time, when no further yielding is observed, to make the arch two-hinged in order to obtain the advantage of increased stiffness.

Making the bottom chord a massive rib sustaining much of the dead load is excellent designing and testifies to the engineer's clear conception of the stiffening function of the truss members of a braced abutment without straining the remaining truss members. This is good mechanics as well as economic design.

The floor lateral truss and the braking girders are additional instances of well-considered design.

The use of 2-in. plates in compression members is extraordinary. The claim made that metal of such thickness showed in tests the same elasticity and ultimate resistance as ½-in. material is contrary to general experience. If the manufacturers were able to roll, of the same material, 2-in. plates of strength equal to the usual structural sizes, it would prove of great interest to the engineering world. Hitherto it has been held that the heavier plate receives less work and shows considerable decrease in strength. Should this claim be substantiated, the use of stitch rivets would decrease considerably, and heavy plates would come into use extensively for columns. This is a matter of much practical importance, and deserves a full discussion by the steel makers.

The "Rules of Design" laid down by Mr. Lindenthal are so interesting that the speaker regrets that they have not been appended in full to the paper.

The three-face joint of the bottom chord is a novel feature in compression chord bearing, and is a move in the right direction to reduce secondary stresses caused by erection deflections. High edge stresses are thereby avoided. The speaker, however, is of the opinion that the concentration of stress on the middle portion should be taken care of in the allowable unit stress. In other words, the computed excess stress should be subtracted from the specified unit stress for the compression members.

The principle adopted by the designer for loads and unit stresses is the right one for long-span bridges. To take into consideration all possible forces and causes, including even an allowance for secondary stresses, and then to fix a proportionately high unit stress is a step in the direction of scientific approximation of actual conditions in the bridge. Of course, it will require much study and consideration to arrive at a final judgment as to the proper unit stress, but, considering all factors, a nearer approach to true conditions will be attained. At the initiative of Mr. Lindenthal, this principle has been partly applied to the Queensboro' Bridge and entirely to the

Manhattan Bridge, and now has been adhered to in the Hell Gate Arch. He deserves credit here for his clearness of vision. Mr. Moisseiff.

A step out of the ordinary is the proportioning of compression members on the basis of net area, deducting the rivet holes. Good reasons for the unusual procedure are brought forward in the paper. The matter, however, is of such extraordinary importance, affecting the design and the economy of all steel structures, that it requires careful research and exhaustive testing to adjust and co-ordinate present standards and specifications. The speaker hopes that the Society's Special Committee on Steel Columns and Struts will give the matter the consideration it deserves.

The speaker has attempted to touch only a few of the most important points in the paper. Many other interesting innovations which it contains are deserving of full discussion.

The paper is well presented, and full of information, and Mr. Ammann deserves the thanks of the Profession for his work.

The Hell Gate Arch Bridge is an excellent example of what engineering genius can accomplish if the project is entrusted to one mind to plan and direct, unhampered by red tape and lay commissioners. The bridge reflects credit on the Engineering Profession, and much praise is due to Mr. Lindenthal and his able associates and co-workers, as well as to the engineers of the American Bridge Company. They have done well.

Samuel T. Wagner,* M. Am. Soc. C. E. (by letter).†—The thanks of the Profession are due to Mr. Ammann for the admirable manner in which he has presented the data of the conception, design, and execution of this most monumental structure. It is possible to find answers to nearly all the questions which arise in its careful reading. This is quite unusual in a work of this magnitude and character. Mr. Wagner.

The use of plates 2 in. thick in built-up members is so out of ordinary practice as to attract special attention. It is hoped that, in closing, Mr. Ammann will see fit to give some of the actual tests of this material. The writer's experience has been that such thicknesses are likely to produce material which is not wholly reliable. Years ago, plates 1 in. in thickness of satisfactory quality were difficult to obtain, especially before the use of open-hearth steel was general. If Mr. Ammann could give the details of the manufacture of these plates, it would be specially interesting. If it is possible to obtain such sections of good quality, even at slightly increased cost, it will open the way for many details which have not been considered possible up to the present time.

The details of the latticing are interesting. There are many advantages in the use of stiff latticing on members of any considerable

* Philadelphia, Pa.

† Received by the Secretary, December 3d, 1917.

Mr. Wagner. size. Many members are designed with such light and inadequate lattice bars that even the most careful handling in the shop and during erection results in damage to them; and, frequently, splendid work in the shop is spoiled before it is placed in the structure. The spacing and general arrangement of the latticing are also to be commended.

The facing of the ends of such large members is a work that requires unusual care in the shop. There can be nothing more important than proper bearing in compression members, and the machine work required, in the writer's opinion, is one of the most difficult parts of the shop fabrication that has to be done on such a structure. It would appear, from the stress measurements made on the completed structure and given in the paper by Mr. Steinman, that this part of the fabrication is specially commendable on the part of the manufacturers. The details of the joints in the bottom chord are most unusual and interesting, and indicate the great care which was taken in planning them. The results undoubtedly confirm the wisdom of the design.

The method of drilling the main members by punching a limited number of holes, using tack-bolts, and drilling the majority of the holes through the solid, is good practice. There is no doubt in the writer's mind, judging from his past experience, that this is the proper way to do work of this character, not only from the standpoint of doing good work, but also of doing it in the most economical and workmanlike manner. It is specially true for the higher carbon steel which was used.

The assembling of the arch in sections is also another most excellent detail of the fabrication. It would be interesting to know whether this was a requirement of the specifications. For arch work, it is doubtful whether such details could have been fabricated satisfactorily in any other way. The writer's first experience on work of this character was on a very small scale in connection with the three-hinged arch trusses of the train-shed of the Reading Terminal in Philadelphia.* In this case, the complete half arches were assembled in the shop. If this had not been done, there surely would have been bad workmanship and difficulty in the erection. It is a great comfort to any manufacturer to know that the joints thus assembled are sure to match. This is especially true in the case of the Hell Gate Arch, with its difficulties in erection.

The use of ballasted floor construction is to be commended, although its cost is greater than the other types referred to in the paper as having been considered. The reasons given for its adoption are well expressed, namely, "more uniform roadbed, less noise and impact, smaller cost of maintenance, and greater safety in case of derailment or fire." Although it seems to be difficult to show in actual figures the desirability of such construction from a financial standpoint, there can be no doubt that under the conditions in this case the additional cost was

* *Transactions*, Am. Soc. C. E., Vol. XXXIV, p. 181.

fully warranted. The advantage of having a standard track in place of special ties is important. Experience in viaduct construction in which a solid floor was used has shown conclusively that it is the best known detail for the prevention of noise. In a number of lawsuits, in which damage from noise has been claimed, no attempt has ever been made to produce proof. It is rather unusual at this time to see a floor of this type designed so that there is sufficient steel to carry the loads without any allowance for the use of the embedding concrete. It is a detail, however, of which the writer approves, especially in such a thin floor. He would have gone one step further, however, in that the floor would have been water-proofed in order to give additional durability to the construction, and it is believed that the additional cost would have been fully justified. If, by some means, water gets through the concrete and to the beams, a condition is created which is very unpleasant to contemplate. After traffic has once been placed on a structure, the difficulties of making any repairs to the floor are very serious. Mr. Wagner.

The officials of the New York Connecting Railroad are to be congratulated on having the foresight and financial courage to design and construct this bridge for four tracks instead of two. It is a monument to all concerned.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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PAPERS AND DISCUSSIONS

STRESS MEASUREMENTS ON THE HELL GATE ARCH BRIDGE

Discussion *

By Messrs. H. J. Bingham Powell, J. A. L. Waddell, F. H. Frankland, L. A. Waterbury, F. D. Hughes, A. H. Fuller, and James E. Howard.

H. J. Bingham Powell,† M. Am. Soc. C. E. (by letter).‡—The writer appreciates the care observed to obtain reliable results in taking the stress measurements on the Hell Gate Arch Bridge, but would question the accuracy of the methods used, that is, to a matter of tenths of a thousandth of an inch claimed. As Inspection Officer in Charge of the Department of Gauges and Standards of the British Ministry of Munitions in the United States, he has had, during the last two years, a wide experience of the difficulties that are encountered in taking fine measurements correct to a tenth of a thousandth of an inch, unless the instruments are extremely accurate and are used in uniform conditions of working, temperature, etc. Mr. Powell.

The principle of measurement of the Howard extensometer appears to the writer to be fundamentally unreliable, because of the following objectionable features: The measuring points of the micrometer caliper are conical, and the zero reading for any given temperature is taken from a "comparison bar" with holes. The member to be measured has, similarly, conical holes for the insertion of the micrometer measuring points. The bearing of the measuring points is not on the tips, but on the sides, and that is where the unreliable feature in the measurement enters. If the micrometer is not held so that the points

* This discussion (of the paper by D. B. Steinman, Assoc. M. Am. Soc. C. E., published in October, 1917, *Proceedings,* and presented at the meeting of November 21st, 1917), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† New York City.

‡ Received by the Secretary, November 1st, 1917.

Mr. Powell. are "square" to the holes, the bearing on the conical sides is very unequal, and, consequently, the measurement is a false one. Further, it is a most difficult matter to get a satisfactory micrometer "feel" with conical points bearing against the inclined sides of the holes. These sources of error present themselves twice: in taking the "zero" reading on the comparison bar, and again when measuring the member. Thus, the combined error may be considerable. Further, the holes in the member are filled with oil, vaseline, or ivory black paint for protection until used. The filling is removed by a pointed aluminum rod, a small cedarwood stick, and some absorbent cotton. No doubt, every care was taken in removing the filling, but in the confined space and under the generally disadvantageous conditions in which the measurements were taken, it would be difficult to obtain the thorough cleaning out of the holes so necessary for any degree of accuracy. Even with the polished surfaces of screw gauges, the writer has found that a film of oil on the thread upsets the measurements entirely; and the conical holes are not only similar in form to the cross-section of the thread of a gauge, but also give a bearing on a surface instead of a line, as in the former case, and thus aggravate the evil of any film of matter present. Also, the holes in the member are of very imperfect finish of surface, and aid the adhesion of such a film. The only way to remove these factors of inaccuracy is to abolish the use of holes and substitute slightly projecting ball-ended cylindrical studs, which present a smooth surface, easily cleaned, and over which very accurate measurements can be made. These studs can be of very small diameter and only project sufficiently for an ordinary micrometer (a micrometer caliper is unnecessary in this case) to have a bearing on the anvil and spindle. The comparison bar of the micrometer would be a plain rod with the ends ground true, and so the zero reading would be a direct, and consequently, exact, one.

Mr. Waddell. J. A. L. Waddell,* M. Am. Soc. C. E. (by letter).†—This paper is not only of great value on account of the important measurements of both direct and secondary stresses which the author records and discusses, but it is also exceedingly interesting because of the historical treatment of the subject of measuring the actual intensities of working stresses in bridge members.

The practice of stating near the beginning of a paper the history of the subject treated is one that is to be highly commended in many cases, for the reason that it places the reader at once *au fait* with the matter under consideration, and thus arouses his interest and induces him to study seriously what follows.

Reliable information concerning the actual intensities of working stresses in main members of bridges, and especially in their connect-

* Kansas City, Mo.

† Received by the Secretary, November 5th, 1917.

ing details, is still rather meager, notwithstanding the fact that such information would be of inestimable value to all designers of steel bridges. The difficulty is that the experiments necessary to obtain the required knowledge are expensive, and are generally beyond the reach of the individual engineer. Engineering societes and railroad companies are the logical organizations for conducting such experiments; and, recognizing this, the American Railway Engineering Association (probably the most active and progressive of all our technical societies), some years ago combined forces with a number of the prominent railroad companies and, by a well-evolved series of experiments, secured a mass of most valuable information on the subject of impact on bridges and bridge members. A similar series of experiments on actual intensities of secondary stresses in bridge members and their connecting details might well be undertaken by the same combination of forces. Mr. Waddell.

The Hell Gate Arch experiments, incomplete as they are, constitute a fine start on the investigation of stress distribution; but they should be completed, so as to include the effects of live loads. Such a finished series of experiments would extend our knowledge of stress conditions, would determine the efficacy of new structural features, would provide reliable information concerning the effects of various methods of erection, and would develop improvements in both design and construction. Again, in this manner, there could be ascertained the relative importance of indeterminate stresses, as well as the actual effects of using redundant members.

Although, as just stated, the scope of the Hell Gate Arch investigation was somewhat limited, the results obtained are certainly valuable; and the engineers who evolved and conducted the series of experiments are certainly entitled to the hearty thanks of the entire Engineering Profession.

F. H. FRANKLAND,* ASSOC. M. AM. SOC. C. E. (by letter).†—This subject is of special importance and interest to all those engineers engaged or interested in the design or construction of important steel bridges, and the author deserves high commendation for his thorough, painstaking, and masterly treatment of a subject acknowledged to be of the highest importance in the development of the science of bridge designing. Mr. Frankland.

In planning the work it is doubtful whether the methods selected and devised could be well improved upon, having in mind the purpose in view. Although the paper represents a tremendous amount of work in the calculation of the secondary stresses in the structure, involving the solution of a large number of simultaneous equations for each condition of loading, and the analytical work required in

* New York City.

† Received by the Secretary, November 2d, 1917.

Mr. Frankland. digesting the results and deducing conclusions, it is remarkable that the work has been presented in such condensed form, and yet is fully adequate for complete understanding. This tendency toward elimination of all extraneous matter in technical papers on special subjects is to be highly commended.

The importance of such stress measurements is so great that it might be safely said that such investigations, in the future, will prove one of the most useful aids in the design of bridges of unprecedented span. The writer is of the opinion that the desirability of more and extended investigations of this nature is clearly indicated; and he begs respectfully to suggest that the Society take under consideration the idea of similar stress measurement investigations on the Quebec and Sciotoville Bridges, and the completion of the measurements on the Hell Gate Bridge (those applying to live load), thus securing exceedingly valuable and complete data on the largest bridges, of the cantilever, continuous truss, and arch types, in existence. Owing to the probable great expense of these proposed investigations, which would be too onerous for any one individual to bear, and to the undoubtedly great value to the Profession, it would appear quite fitting and proper for the American Society of Civil Engineers to undertake this work, under the supervision of a special committee. It may be well to mention, in elaboration of the foregoing suggestion, that the determination of the live-load stresses of the Hell Gate Bridge would complete the measurements for that structure, and that initial stress measurements have already been made for the Sciotoville Bridge, under the direction of Gustav Lindenthal, M. Am. Soc. C. E. The results deduced from stress measurements of three types of long-span bridges would be more valuable and conclusive than those for a single structure.

Mr. Waterbury. L. A. Waterbury,* M. Am. Soc. C. E. (by letter).†—On page 1824‡ of Mr. Steinman's paper, the following paragraph, which forms the topic of this discussion, appears:

"The foregoing discussion of results does not include the final stage (2-H). In that stage, in the members near the ends of the span, the upper end in every case was found to present a larger average stress than did the lower end. The differences ranged from 450 to 2 800 lb. per sq. in. This anomalous result appears to arise from some unexplained disturbance of stress distribution, and does not represent any error of observation."

This interesting condition of observed stress may possibly be due to an irregular distribution of the stresses at the observed sections, which might be due to the order in which the drift-pins and bolts were

* Tucson, Ariz.

† Received by the Secretary, November 15th, 1917.

‡ *Proceedings,* Am. Soc. C. E., October, 1917.

replaced by rivets. The great size of the members and the great difference between the stress at the center of member 0-2 from that at the top and bottom, for each section, for Stage (*2-H*), suggest that there might easily be considerable irregularity in the distribution of stresses at each section, between the lines of gauged deformations. Mr. Waterbury.

To illustrate the principle involved, consider the simple case indicated by the joint shown in Fig. 7, and suppose that the stages of removal of drift-pins and of placing of rivets are as follows: (1) center line of member *A*; (2) outside rows of member *A*; (3) intermediate rows of member *A*; (4) intermediate rows of member *B*; (5) outside rows of member *B*; and (6) center line of member *B*. During each stage a slight closing of the butt joint occurs, which, for joints of the type used in the Hell Gate Arch, develops larger stresses in the middle third of each web than in the outer thirds. At the end of Stage (1), the rivets in place are without shearing stress, but during Stage (2) a slight closing of the joint occurs, producing some stress in the rivets placed during the first stage. During Stage (3), the rivets placed in the second stage receive some stress, and those placed in the first stage receive an additional increment of stress, but the two intermediate rows of member *A* are without stress. During Stage

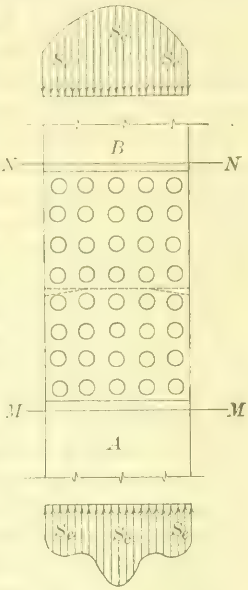

FIG. 7.

(4) a portion of the load which had been carried by the splice plates, is transferred to the web as the butt joint closes, thus relieving the center and outside rows of member *A* of some stress, but, at the same time, developing some negative shear in the rivets of the two intermediate rows of that member. During Stages (5) and (6), the action on the rivets of member *A* is similar in character to that which occurred during Stage (4). At the end of Stage (6), the center row of rivets of member *B* is without stress, the outside rows of the same member have some stress, and the two intermediate rows have considerable stress. Such a distribution of the load among the rivets of the splice would tend to produce a variation of the stress at Section *M-M* of member *A* of the type indicated by the stress profile, which is shown below the joint in Fig. 7, while the variation at Section *N-N* of member *B* would be more nearly like that indicated by the stress profile shown above the joint. If the deformations are measured along lines coresponding to the stresses, S_c and S_e, of the stress profiles, it is evident that the total stress of member *A* may be equal to the total

Mr. Waterbury. stress of member B, in spite of the fact that all the observed stresses for the first member are greater than those for member B. The deformations for member 0-2 for the 2-H stage of erection, as stated in Table 2 of the paper, exhibit this character of variation, notwithstanding the fact that the observed values are large enough to be considered fairly reliable.

In attempting to check the average stress at sections where there is an irregularity in the distribution of the load, it should be remembered that, even if readings are observed at enough lines to determine the correct variation in deformation for the section, the average stress will not necessarily be indicated directly by the stress profile, due to the variation in thickness of metal across the section. The correct average stress would be obtained by weighting the stress for an increment length of profile, the weight being proportional to the area of metal over which that particular stress is effective, and by computing the weighted mean of all the stresses shown by the stress profile.

Mr. Hughes. F. D. HUGHES,* M. AM. SOC. C. E. (by letter).†—Engineering has been correctly called the creative profession, and yet the engineer, who is most directly connected with production of the world's wealth, such as manufacturing and kindred lines, has very little time to devote to independent investigation of engineering problems, and, for that reason, he, more than any other, owes thanks to the authors of this and similar papers, who spend their time in investigation on the frontier of engineering thought and experimental research. The average Board of Directors of corporations looks with scant favor on expenditure of either time or money in search of information and formulas in engineering that do not promise an adequate financial return to the corporation which it represents. Therefore, as already stated, this large body of the Engineering Profession, which, for want of a better term, the writer would designate as industrial engineers, owes much to those who take the time and opportunity to investigate, for the benefit of the Profession at large, such problems as were presented in the construction of the Hell Gate Arch Bridge and to give the results to the engineering world. It is permitted to few engineers to be connected with either the design or construction of a structure of this magnitude, and perhaps the conclusions to be reached are not of practical use to a majority of the Profession, and yet the paper, in itself, is a very valuable contribution to engineering literature.

In an experience of some twenty years, mostly devoted to the design and construction of highway bridges, the writer has found a wide diversity of treatment in the consideration of secondary stresses. Such treatment has varied from that degree of refinement which might be termed "painful" in its exactitude, to that in which the stresses were

* St. Louis, Mo.

† Received by the Secretary, November 21st, 1917.

Mr. Hughes.

completely ignored. As an instance, he recalls the construction of a 140-ft. span, designed to carry a concrete floor of the Warren subdivided panel type, all members being riveted, in which the wind stress was carried into the lower chord and exact consideration was given to the increment of stress, even to the changing of the size of section in adjoining panels where the live- and dead-load stresses were of equal moment, thus necessitating (if the theory of the design is followed), a field connection between the two panels where practice should have made the two in one single member. In view of the fact that the stringers were rigidly connected to the floor-beam, the writer considered this degree of refinement a waste of material. On the other hand, he has seen numerous structures of highway design, of the pin-connected type, in which the entire lower chord was composed of eyebars, and in which the first vertical member consisted of two light angles barely sufficient to carry the figured live- and deal-load stresses; and the detail of connecting the floor-beam below the chord, would, by the same analysis of wind stress, cause bending in the vertical member of from 40 000 to 50 000 lb. per sq. in.; and yet these bridges are standing and have been carrying ordinary traffic for years. Between these two extremes lies the most acceptable practice, and the author's conclusion, that any treatment of secondary stresses should be carefully analyzed and the experiments along the line of stress measurements taken with considerable allowance, is very timely.

The lesson to be learned from the result of these measurements, it seems to the writer, is, that all structures which are designed to carry their own weight during erection, as well as eccentric loading from machinery or other methods of handling, should have some scheme of measurements for checking the secondary stresses. The additional expense would be more than made up by the insured safety of the structure against probable loss of life and limb, and would have prevented numerous disasters which have occurred on structures of this class.

Mr. Fuller

A. H. Fuller,* M. Am. Soc. C. E. (by letter).†—The Engineering Profession is certainly indebted to Mr. Lindenthal for developing the erection of the Hell Gate Arch Bridge into a huge, scientific experiment, and to the author for the admirable manner in which this has been carried out and presented. No one who has given any attention to stress measurements can fail to appreciate the precautions taken in this work and the care in carrying them out, and neither can any one who has ever attempted to compute secondary stresses fail to recognize the magnitude of the task along this line. Each of these features could well be considered an accomplishment, and the combination of the two, on a structure of the magnitude and interest of the one under

* Easton, Pa.

† Received by the Secretary, November 22d, 1917.

Mr. Fuller. consideration, marks a new achievement in the understanding of the actual behavior of steel structures.

Engineers would not be doing their duty by simply applauding what has been done without making an effort to analyze the results, in a manner that would throw even greater light on the subject, and to remove the last shadow of doubt in regard to the interpretation that may be drawn.

On page 1830,* it is stated that the following relations are evident:

"1.—Except for the smaller percentage of secondary stresses, the measured values are consistently lower than the calculated values.

"2.—As the percentage of calculated stresses increases, the percentage of measured values also increases, but not as rapidly.

"3.—For the highest percentages of secondary stresses, the measured values amount to only a small fraction of the calculated values."

On page 1836,* in Paragraph 5 of the Summary of Conclusions, the same points are brought out. A probable interpretation from these conclusions is that measured secondary stresses are usually lower than computed ones.

An examination of Plate XXXVI discloses the fact that the recorded measured secondary stresses are as great as, or greater (frequently much greater) than, the computed ones in all cases where the primary unit stresses exceed 5 000 lb. per sq. in. and, with one exception, for all cases where the primary unit stresses exceed 3 000 lb. per sq. in., that is, that the measured stress is greater than the computed one whenever the primary stress approaches a maximum working value.

Although the author withholds the measured secondary stresses for the stage, 3-*H*, because of the inconsistencies due to added dead load during observation, the primary stresses are given. It seems improbable to the writer that this disturbance could be so great as to render the results valueless, and he would like to ask the author for these results in connection with the points that have just been raised.

On Plate XXXIX are given the percentages of computed secondary stresses in all members, and it is of interest to note, as the author has so well brought out, that these are remarkably low in the lower chord members which carry the greater portion of the load. Take Member 4-6, for instance, in which the computed secondary stress due to dead load is 5% of the primary stress. Plate XXXVI indicates that the measured secondary stress is 13% in one truss and 11% in another, of the primary stresses, that is, more than double the computed ones. The writer would like to ask what information is available concerning the variation of stresses within the 20-in. gauge line that was used. The intensity is certainly greater toward the end of

* *Proceedings*, Am. Soc. C. E., October, 1917.

the member. Measurements that have been taken by the writer on steel and concrete in reinforced concrete structures indicate a very rapid drop in maximum stresses from the point of connection of the member, and suggest that gauge lengths even less than the usual 8 in. are needed to catch the maximum stresses. This point has been brought out much more clearly by Professor McMillan, of the University of Minnesota, in comparisons between 2-in. and 8-in. gauge lengths. It seems, therefore, that the intensity of secondary stresses may be somewhat greater than the averages over 20-in. gauge lengths, and that they are intensified again at the point of decrease in section at the edge of the gusset plates. Mr. Fuller.

These points can be recognized without questioning the author's general conclusions that the secondary stresses in the structure under consideration are not large enough to cause concern, and that the type of structure is well chosen to keep them within low values. It does not seem to the writer that it is well to minimize these stresses, for that would tend to give unwarranted confidence and possibly result in overlooking secondary stresses and stress measurements in cases where they might otherwise be secured and thus contribute to the store of knowledge.

The writer would like to ask for more information than has been given in the paper, concerning the distribution of stresses in different portions of the members. Easy computations from the data in Table 2 show the average intensity of the stress in the central angles to be about 50% greater than the average intensity at the four extreme angles for the 2-*H*, or last, stage, for the upper end of Member 0-2. Similar conclusions may be drawn from Table 4 for the upper end of Member 4-6. The author has well pointed out that the nature of the connections tends to concentrate the stress in the middle portion of the member, but presents only average values, except for the data in Tables 1 to 4. It would be interesting to have any data that may be available for particular locations in regard to the distributions of stress between plates and angles. The writer would not anticipate any great change in work so carefully designed and constructed as this has been, but feels that the information would be desirable.

On page 1816,* the author states that the Howard strain gauge is probably the simplest and most accurate field instrument for measuring strains. As the writer has never used this instrument, he is not in the position to question the statement. He is under the impression, however, that more information may be secured from the quicker acting Berry strain gauge for the same expenditure of time and energy. The inference drawn by the writer from the work on the Hell Gate Arch Bridge is that for each load only one measurement was recorded for each gauge length. Possibly this has been verified from

* *Proceedings*, Am. Soc. C. E., October, 1917.

Mr. Fuller. several readings and the average recorded. The writer has secured much more consistent results from the Berry instrument by taking three or more sets of readings, with change of dial between each set, and recording all the results, than by taking all the readings at a certain point consecutively and recording an average.

Mr. Howard. JAMES E. HOWARD,* ESQ. (by letter).†—This is a most interesting and instructive paper on the results of measured strains on a structure of magnitude, which, in its conception and execution, occupies the foremost rank in engineering works. The intelligent care which was exercised in acquiring the experimental data, the impartial criticism of this method of measurement, the definition of its proper scope, as well as its limitations, is a source of gratification to the writer, who believes himself to be the one who inaugurated this simple and direct method of examination, a method which is capable of furnishing many useful results which have not yet been placed before the Engineering Profession.

This method of measured strains has been used by the writer in the examination of the dead-load strains in bridges of large span, in the structural members of modern buildings, in the distribution of stresses in the sheets and across the seams in steel boilers, in the investigation of the internal strains in steel rails, and also in the study of thermal effects in street pavements. On dates earlier than the examples just mentioned, the method was used in ascertaining the state of internal strains in steel forgings, in railway axles, in cold-rolled shafting, and also the residual strains in steel bars after over-straining loads had been applied and released.

This method of examination was first used by the writer, in 1886 and 1887, in the determination of the internal strains in steel forgings, in lieu of a method used by Gen. Nicholas Kalakousky, of the Russian Artillery, who utilized a microscopic cathetometer for the purpose.

The use of drilled and countersunk holes for defining the gauged lengths enabled such lengths to be established and preserved against accidental injury, even against vicissitudes of street traffic, in the examination of street pavements; in structural members, protection is afforded to the contact surfaces during handling and erection, in addition to which the feature of permanence is present, enabling a re-examination of the gauged lengths after the lapse of time.

A reference bar, having substantially the same coefficient of expansion as that of the work under examination, serves its purpose as a standard of length. The reference bar is kept at the same temperature as the work, whenever it is practicable to do so, at other times a correction for the difference in temperature is applied.

* Washington, D. C.

† Received by the Secretary, November 26th, 1917.

Mr. Howard.

Two types of strain gauges—or transfer instruments, as they might be termed—have been used: one is adjustable to any gauged length from 1 in. upward, 36 in. being considered a practical maximum, the other being represented in the type used by the author. The adjustable gauge was the earlier design. It was intended for use on horizontal surfaces or those nearly so, and is more essentially a laboratory instrument than the type used on the Hell Gate Arch Bridge. The latter admits of being used in different positions.

The subject of measured strains is one which is particularly attractive to the writer, as it appears to open the way to advance knowledge relating to the actual conditions or states of strain which pertain to engineering structures of all kinds. Dead-load strains, equivalent to 20 000 lb. per sq. in., have been observed in bridge members. Temperature differences have been responsible for stresses of several thousand pounds per square inch in large members. In steel rails, wheel pressures have caused the introduction of internal stresses, exceeding 20 000 lb. per sq. in., in the head of the rail next the running surface, with tensile components in the interior of the head, stresses which, singularly, appear to have been overlooked in railway engineering.

It is felt that a most promising opportunity for the acquisition of engineering data has been permitted to pass unimproved, or has been utilized to a limited degree only, in the few examples of measured strains in engineering structures. It is with great pleasure, therefore, that the writer witnesses the presentation of this valuable paper which was provided for by Gustav Lindenthal, M. Am. Soc. C. E., the Chief Engineer of the magnificent Hell Gate Arch Bridge.

MEMOIRS OF DECEASED MEMBERS

NOTE.—Memoirs will be reproduced in the volumes of *Transactions*. Any information which will amplify the records as here printed, or correct any errors, should be forwarded to the Secretary prior to the final publication.

DANIEL MARSHALL ANDREWS, M. Am. Soc. C. E.*

DIED JUNE 28TH, 1917.

Daniel Marshall Andrews was born in Americus, Ga., on October 24th, 1853. He was the son of Judge Garnett Andrews and Annulet Ball Andrews, of Washington, Ga.

In the spring of 1872 he entered the Engineering School of Georgia University, at Athens, Ga., and would have been graduated in 1874, had not a serious illness prevented him from taking the final examinations. In later years his Alma Mater issued his diploma and conferred on him the degree of C. E., as of the class of 1874. This action was based on his excellent record as a student, and his subsequent high standing in the profession of Civil Engineering.

Mr. Marshall left college in the midst of the panic of the Seventies, and, after trying unsuccessfully for six years to grow 5-cent cotton at a profit, he, in 1881, returned to his profession.

From 1881 to 1884 he was employed on the survey, location, and construction of railroads in Georgia and South Carolina, embracing certain lines of the Seaboard Air Line System in these States. From 1884 until his death he was engaged in river improvements with the United States Engineer Department, except for a short time in 1902 when he investigated and prepared estimates of cost for the Galveston sea wall. While in the Government service, Mr. Andrews' work consisted of the design and construction of navigation locks and dams, the regulation of rivers, the investigation of many Southern rivers, together with the storage reservoirs on their headwaters, with a view to the co-ordination of power and navigation, and, at the time of his death, he had charge of the improvement of the Coosa River and its tributaries in Georgia and Alabama, the Alabama River in Alabama, the Chattahoochee River in Georgia and Alabama, the Flint River in Georgia, and the Apalachicola and Choctawhatchee Rivers in Alabama and Florida.

He was a member of the following scientific and technical societies: The American Association for the Advancement of Science; the American Historical Association; the International Navigation Congresses; the National Geographic Society; the Alabama Anthropological Society; the Bartram Society of Natural History; and the Alabama Technical Association.

* Memoir prepared by B. M. Hall, M. Am. Soc. C. E.

The following is a partial list of technical and historical papers written by Mr. Andrews: "The Economic Improvement of the Coosa and Alabama Rivers in Georgia and Alabama",* "Backwater Above Dams",† "Foundations in the Coosa and Black Warrior Rivers",‡ "Dam No. 5, Coosa River, The Problems of Location and Construction",‡ "The Improvement of Non-Torrential Streams",‡ and "The Route of DeSoto from Cofitachequi in Georgia to Cosa in Alabama", read before the Alabama Anthropological Society.

Although Mr. Andrews was a U. S. Assistant Engineer for the greater part of his life, engaged in engineering works of considerable magnitude, he was also a great student of science and general literature.

On April 20th, 1897, he was married to Miss Adeline Van Court, of Natchez, Miss., who survives him, together with their three sons, Daniel Marshall, Jr., Van Court, and Garnett.

At the time of his death, and for many years previous, his home was at Montgomery, Ala. In June, 1917, he went to Chicago, Ill., for an operation on his throat, and died there on the 28th of that month.

Mr. Andrews, known familiarly by his boyhood friends as Marsh Andrews and by the general public as Major Andrews, was a Christian gentleman, a man of integrity and honor, and an able engineer of scholarly attainments. He was a man of positive convictions, dignified bearing, and a delightful personality. He was devoted to his home, his family, and his friends and was enthusiastic in his professional work.

He was elected a Member of the American Society of Civil Engineers on March 2d, 1892, and always took an active interest in its affairs.

WILLIAM SINCLAIR BACOT, M. Am. Soc. C. E.§

Died October 31st, 1917.

William Sinclair Bacot, the son of Robert Cochran Bacot and Mary Gilchrist Bacot, was born in East Orange, N. J., on April 19th, 1860. He was a direct descendant of Pierre Bacot, a French refugee who fled to America after the Revocation of the Edict of Nantes and settled in Charleston, S. C., in 1685.

Mr. Bacot was prepared for college at Hasbrouck's Institute, in Jersey City, N. J., and entered Princeton College in 1877, from which he was graduated with the class of 1881, with the degree of C. E.

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* *Transactions,* Am. Soc. C. E., Vol. L, p. 363.

† *Engineering News.*

‡ *Professional Memoirs,* Corps of Engrs., U. S. A.

§ Memoir prepared by John V. Bacot, Esq., Utica, N. Y.

After leaving college he was first employed as an Assistant Engineer on the construction of the Hackensack Water Company's Works in New Jersey. From 1882 to 1908, he was engaged in private practice, having an office in New York City and making a specialty of improved highway building and hydraulic engineering and construction.

Mr. Bacot was appointed by the United States Government to build the Good Roads Exhibit at the World's Fair at Chicago. For several years he was County Engineer of Staten Island, New York, and built good roads in that place and also at Lenox, Mass., and Burlington, Vt.

Among the water-works constructed and operated by Mr. Bacot were those at Fishkill, Portchester, Matteawan, and Mount Vernon, in New York State, and Greenwich, Conn. His last work was the development of the water-works for the City of Utica, and the surrounding towns and villages, covering the period from 1900 to the present time. He designed and built the West Canada Creek supply system for Utica, and welded into one the water-works of Utica, New Hartford, Whitestown, Oriskany, and other adjoining towns and villages.

At the time of his death, Mr. Bacot was President of the Consolidated Water Company of Utica, N. Y., and its Chief Engineer, positions which he had held almost continuously for many years.

He was of a retiring disposition, although genial and of a lively and enthusiastic temperament, inheriting the latter characteristics from his French ancestry.

In May, 1917, he was married to Anne Harrison Hendryx, of Cincinnati, Ohio, a great-granddaughter of the late President William Henry Harrison.

He is survived by his brothers, John V. and Richard Wainwright Bacot, and a sister, Mrs. Annie B. Roundey, and several nieces and nephews.

Mr. Bacot was elected a Member of the American Society of Civil Engineers on October 1st, 1890.

ELMER ELLSWORTH COLBY, M. Am. Soc. C. E.*

DIED SEPTEMBER 27TH, 1917.

By the death of Elmer Ellsworth Colby, the Engineering Profession loses a man of merit and distinction and his associates a friend in every sense of the word. He was born in Oconomowoc, Wis., on July 2d, 1861, and was educated in the public schools of Wisconsin, Iowa, and Illinois. He was graduated from a Chicago High School in 1879.

Mr. Colby's professional life commenced in his nineteenth year

* Memoir prepared by Max L. Cunningham, M. Am. Soc. C. E.

when he began service as a Rodman on the construction of a Chicago, Milwaukee, and St. Paul Line in Carrol County, Iowa. His advancement was rapid, for, in March, 1883, he was made Division Engineer of the Kansas City, Springfield, and Memphis Railway, in charge of construction on the Memphis Division, Terminal Yards, and the Mississippi River Transfer.

After the termination of this work in 1884, and until the latter part of 1888, Mr. Colby was engaged in various construction and locating capacities in Iowa, Missouri, and Kansas, changing occupation frequently, as was the custom at that time, but always advancing, and broadening his experience. In November, 1888, however, he began work in that branch of his Profession which was destined to be his life's work, and for which he was best known among his later associates.

At that time he opened an office for the private practice of bridge designing and construction in Springfield, Mo., acting at the same time as County Engineer for Greene County, Missouri. As was to be expected, the first years of such an enterprise were a bitter struggle, and it was necessary for him to do land work, sub-division surveys, and any work of that nature that would come to a young engineer starting his own business at such a time, but he steadily continued toward his goal, and built, during this period, several bridges across the larger streams in the mountainous portion of Missouri, and, also, after entering a competition, the War Department awarded him the contract for designing and supervising the construction of a road from Springfield to the National Cemetery. He also built the unique Massey Building in Springfield, a four-story structure carried on a stone arch spanning Wilson Creek, and designed and built the concentrating mills and shafts for several lead and zinc plants, as well as doing prospecting and underground work for the companies.

When, in 1900, the United States Department of the Interior started the work of platting segregated town sites in the Chickasaw Nation, in Indian Territory, Mr. Colby was placed in charge of the work, and moved to Chickasha, where he resided until his death. After the town site work was completed, he began again his private practice, and, during 1903, was Chief Engineer of the Colorado, Oklahoma, and Texas Railway, a Santa Fé property connecting the main north and south lines with the main line to the Southwest through the State of Oklahoma.

The private practice opened by Mr. Colby in Chickasha included the supervision by him, as City Engineer, of all public work in that city, covering the expenditure of approximately $1 500 000. He also acted as Consulting Engineer for other cities in Oklahoma, on various municipal improvements, notably sanitary and paving work.

At the time of his death, Mr. Colby was also County Engineer for Caddo and Grady Counties, Oklahoma. He had recently completed

a most important and difficult bridge across the South Canadian River, at Canadian, Tex., and his services as an expert on construction and cost were in great demand.

He was instrumental in securing the present highway and sanitary laws of Oklahoma, and was one of the founders of the Oklahoma Society of Engineers, having served as its President in 1910. He belonged to the B. P. O. E., and was also an active member of the American Society of Municipal Improvements and of the American Association for the Advancement of Science.

Professionally, Mr. Colby was a marked man in his community, and his interest in civic affairs made him a leader, both in Chickasha and in the State. His death was hastened by application, as a citizen, to a matter that was absolutely valueless to him as an engineer or to his business. The incident was the culmination of a series of public services extending over many years and appreciated more after his death than before it.

He leaves a widow, two sons, both of whom are Civil Engineers—the younger of whom is taking up his father's work so far as he can—six daughters, and a host of friends, all of whom feel his loss as a personal one. He died at work, having been stricken by heart failure while at the wheel of his car on his way to inspect some work in his charge. His last words were an inquiry as to whether he was "on the right road."

It is difficult for the writer to express his feelings at the loss of a friend such as Mr. Colby had been for years, but it seems to him that any engineer who can live as full and useful a life as Mr. Colby had, who can become so valuable a man to the State, and can be so universally mourned, has accomplished his life's work in an enviable manner and could well be proud to die in harness with his hand to the wheel as he did.

Mr. Colby was elected a Member of the American Society of Civil Engineers on January 7th, 1913.

HARRY MADERA GOULD, M. Am. Soc. C. E.*

DIED SEPTEMBER 30TH, 1917.

Harry Madera Gould, the son of John E. and Cora D. Gould, was born at Worthville, Ky., on April 12th, 1872. He received his grammar and high school education at Worthville and Louisville, Ky., and was graduated from Rensselaer Polytechnic Institute, in the class of 1895, with the degree of Civil Engineer.

He entered railroad work with the "Big Four" Railroad at Louisville, Ky., but moved to Colorado, where he was engaged for a while in

* Memoir prepared by C. B. Wilson, M. Am. Soc. C. E.

mining work. In 1897, he was employed by the Louisville and Nashville Railroad Company as Assistant Engineer. From 1898 to 1905, he was associated with his father in the erection of steel bridges, principally railroad structures, and, from 1906 to 1908, he was Secretary and Treasurer of the Gould Construction Company, doing the same class of work. In 1907 and 1908 the Gould Construction Company, in conjunction with the Foster and Creighton Company, erected the two steel and concrete highway bridges over the Cumberland River at Nashville, Tenn., and Mr. Gould was General Manager of the work. From 1909 to 1914 he was Vice-President and General Manager of the Foster-Creighton-Gould Company, during which time the company built the Kentucky and Indiana Railroad bridge foundation in the Ohio River, at Louisville, erected the Fort Smith and Van Buren Highway Bridge superstructure across the Arkansas River, built the Louisville and Nashville Railroad Bridge complete across the Cumberland River, at Nashville, and a number of railroad bridges and other structures. From 1915 to 1917 he was President of Gould Contracting Company, doing steel erection and foundation work, having just completed a power dam across Caney Fork River, Tennessee; he had also been engaged on the construction of the Hydes Ferry Bridge over the Cumberland River, at Nashville, Tenn., now nearing completion.

Mr. Gould had been in bad health for several months and died at Nashville, Tenn., on September 30th, 1917. His wife and one child survive him.

He had been a member of the Engineering Association of the South (now the Engineering Association of Nashville) since May, 1903, and had served as a Director and as President. He was a man of high business principles and ability, whom not only his friends, but the community, will miss.

Mr. Gould was elected a Member of the American Society of Civil Engineers on November 30th, 1909.

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